ACI 318-19

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Changes to the Concrete Design Standard





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Questions related to specific materials, methods, and services will be addressed at the conclusion of this presentation.

The American Institute of Architects has approved this session for 7.5 AIA/CES LU/HSW Learning Units.



Learning Objectives

1. Understand where higher grades of reinforcement are accepted and changes to the requirements for structural concrete to allow the higher reinforcement grades, including development lengths and phi-factors.
2. Identify the added requirements to address shotcrete as a concrete placement method.
3. Explain the expanded scope of deep foundation provisions, including seismic requirements.



Learning Objectives

4. Learn the new requirements for post-installed screw type anchors and shear lug design for anchoring to concrete.
5. Describe the changes to shear design provisions and equations.
6. Identify new tension longitudinal reinforcement requirements in special structural walls



Speakers

Speaker bios are included in your handouts for the presentation

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Introduction



Today's Seminar

- Major changes
- Grouped by topic

  - Organization
  - Existing structures
  - Loads & analysis
  - Slabs
  - Post-tensioning
  - Precast/Prestressed
  - Circular sections
  - Walls
  - Foundations
  - Anchorage to concrete
  - Seismic

Today's Seminar

- Major changes
- Grouped by topic

  - High-strength reinforcement
  - Development length
  - Shear modifications
  - Durability and materials
  - Strut-and-tie method
  - Shotcrete
  - Appendix A

Today's Seminar

- Changes from ACI 318-14 to ACI 318-19

318-14  318-19

Why Do We Change ACI 318?

- **Reflects new research**
- **Construction practices change**
- **Sometimes tragic events provide introspect**
  - Earthquakes or other natural disasters
  - Collapses or construction accidents
  - Observed in-service performance
- **New materials**
  - Or better ways of making established materials
- **More powerful analytical tools**



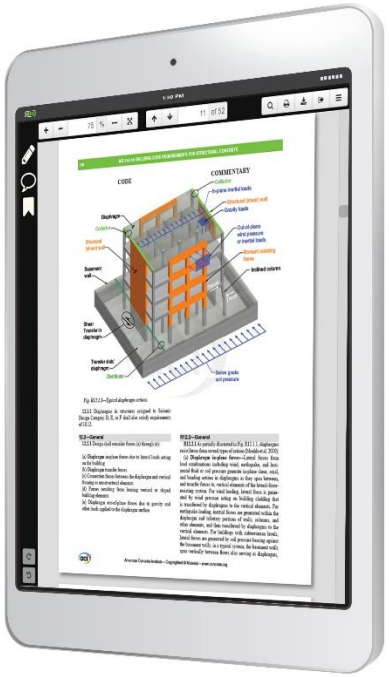
Resources

- ACI 318
- Speaker notes
- ACI Reinforced Concrete Design Handbook
- ACI 318 Building Code Portal

ACI 318-19

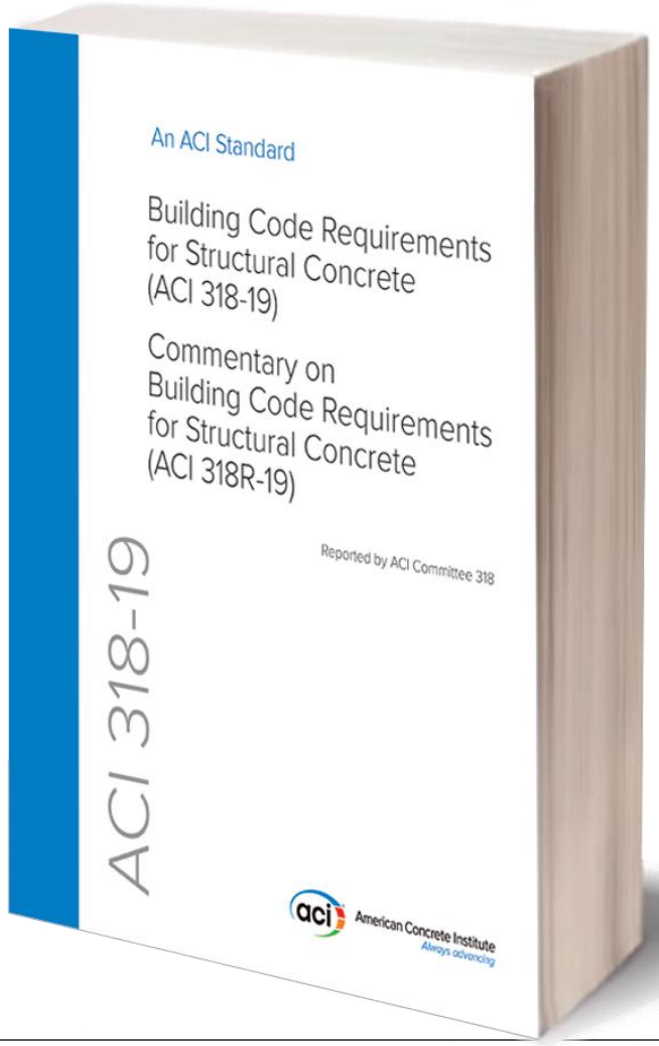
Variety of formats, including:

- **Printed copy**
  - Softcover and hardcover
- **Enhanced PDF**



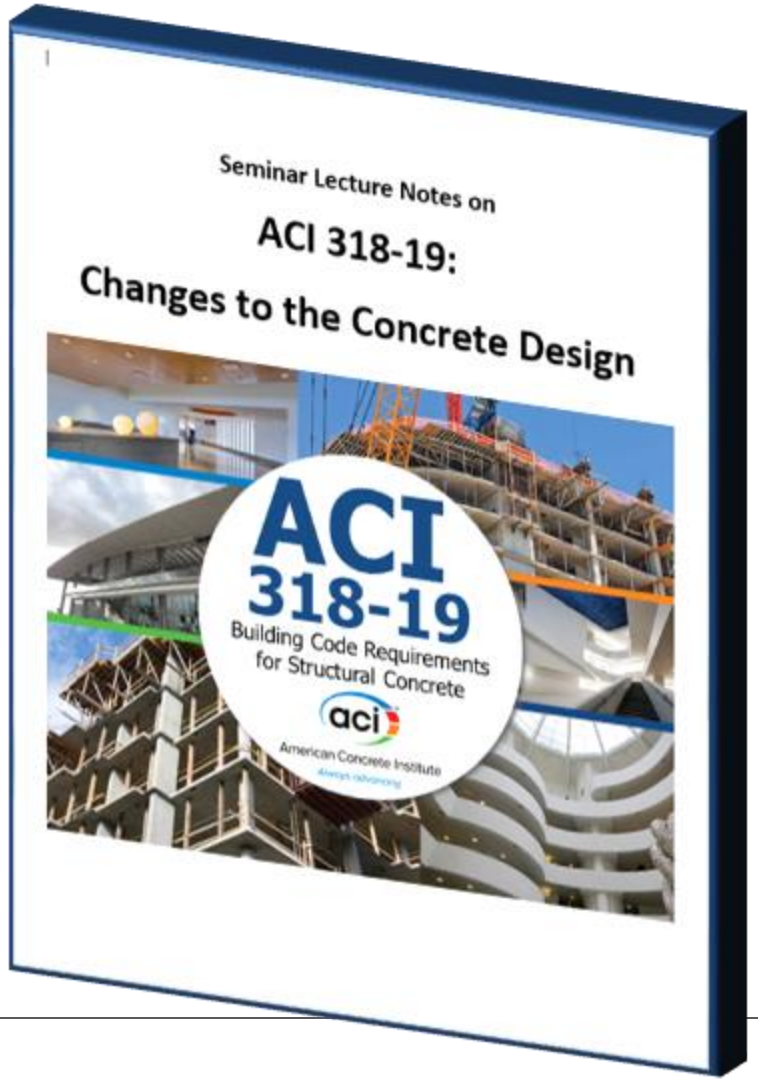
Versions

- **English**
- **Spanish**
- **In.-lb units**
- **SI units**



Speaker Notes

Today's presentation



ACI Design Handbook

- 15 chapters
- Explanatory text
- Design aids
- 2019 version expected early next year



ACI Design Handbook

- 1: Building Systems
- 2: Structural Systems
- 3: Structural Analysis
- 4: Durability
- 5: One-Way Slabs
- 6: Two-Way Slabs
- 7: Beams
- 8: Diaphragms
- 9: Columns
- 10: Walls
- 11: Foundations
- 12: Retaining Walls
- 13: Serviceability
- 14: Strut-and-Tie
- 15: Anchorage

ACI 318 Building Code Portal



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Organization



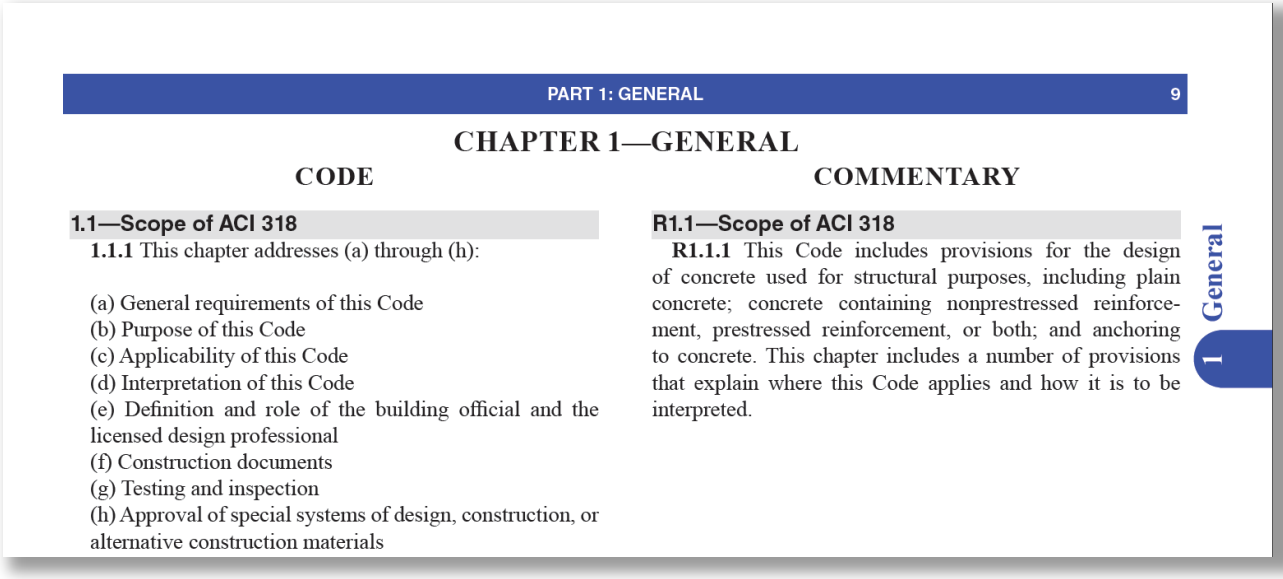
Major goals of ACI 318 organization

- **Ease of use**
- **Find the information you need quickly**
  - Consistent organization
  - Organized in the order of design
- **Increase certainty that a design fully meets the Code**
  - A chapter for each member type
  - All member design provisions in one chapter

Navigation

10 Parts

- General



PART 1: GENERAL 9

CHAPTER 1—GENERAL

CODE

1.1—Scope of ACI 318

1.1.1 This chapter addresses (a) through (h):

(a) General requirements of this Code
(b) Purpose of this Code
(c) Applicability of this Code
(d) Interpretation of this Code
(e) Definition and role of the building official and the licensed design professional
(f) Construction documents
(g) Testing and inspection
(h) Approval of special systems of design, construction, or alternative construction materials

COMMENTARY

R1.1—Scope of ACI 318

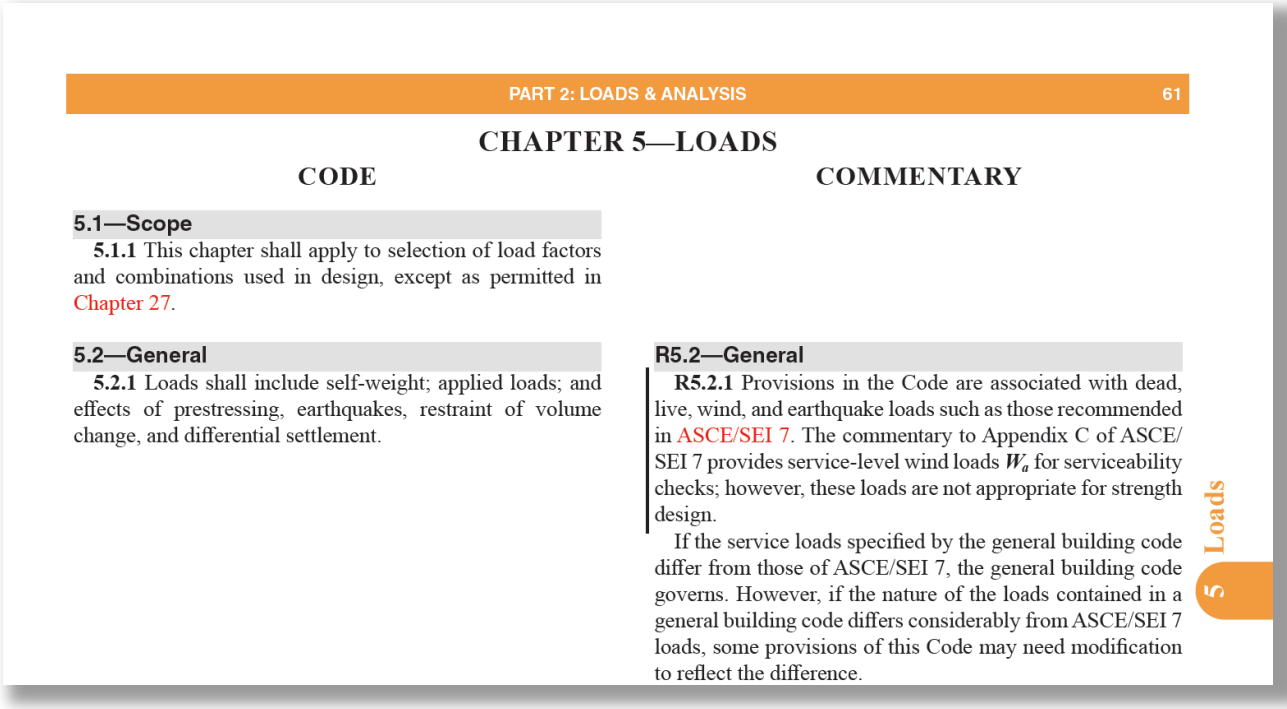
R1.1.1 This Code includes provisions for the design of concrete used for structural purposes, including plain concrete; concrete containing nonprestressed reinforcement, prestressed reinforcement, or both; and anchoring to concrete. This chapter includes a number of provisions that explain where this Code applies and how it is to be interpreted.

1 General

Navigation

10 Parts

- General
- Loads & Analysis



PART 2: LOADS & ANALYSIS 61

CHAPTER 5—LOADS

CODE

5.1—Scope

5.1.1 This chapter shall apply to selection of load factors and combinations used in design, except as permitted in Chapter 27.

5.2—General

5.2.1 Loads shall include self-weight; applied loads; and effects of prestressing, earthquakes, restraint of volume change, and differential settlement.

COMMENTARY

R5.2—General

R5.2.1 Provisions in the Code are associated with dead, live, wind, and earthquake loads such as those recommended in ASCE/SEI 7. The commentary to Appendix C of ASCE/SEI 7 provides service-level wind loads W_a for serviceability checks; however, these loads are not appropriate for strength design.

If the service loads specified by the general building code differ from those of ASCE/SEI 7, the general building code governs. However, if the nature of the loads contained in a general building code differs considerably from ASCE/SEI 7 loads, some provisions of this Code may need modification to reflect the difference.

5 Loads

ACI 318 Style

9.6—Reinforcement limits

9.6.1 *Minimum flexural reinforcement in nonprestressed beams*

9.6.1.1 A minimum area of flexural reinforcement, $A_{s,min}$, shall be provided at every section where tension reinforcement is required by analysis.

9.6.2 *Minimum flexural reinforcement in prestressed beams*

9.6.2.1 For beams with bonded prestressed reinforcement, total quantity of A_s and A_{ps} shall be adequate to develop a factored load at least 1.2 times the cracking load calculated on the basis of f_r defined in 19.2.3.



Navigation

10 Parts

- General
- Loads & Analysis
- Members
- Joints/Connections/ Anchors
- Seismic
- Materials & Durability
- Strength & Serviceability
- Reinforcement
- Construction
- Evaluation

Part 1: General

- **1: General**

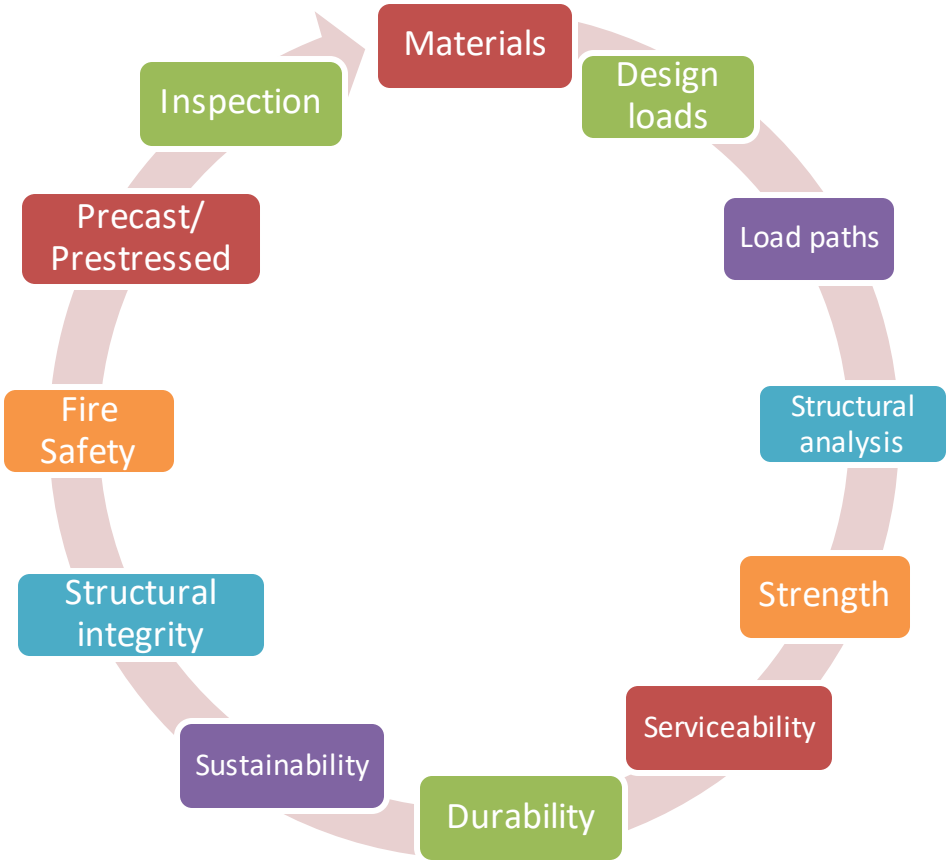
- **2: Notation and Terminology**
  - d_{agg} = nominal maximum size of coarse aggregate, in.
  - aggregate—granular material, such as sand, gravel, crushed stone, iron blast-furnace slag, or recycled aggregates including crushed hydraulic cement concrete, used with a cementing medium to form concrete or mortar.

Part 1: General

- 3: Referenced Standards

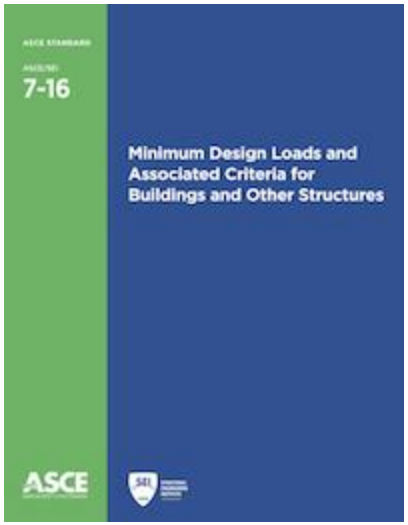
ASTM INTERNATIONAL

- 4: Structural System Requirements



Part 2: Loads & Analysis

- **5: Loads**



- **6: Structural Analysis**
  - Simplified, first-order, second-order
  - Linear, nonlinear
  - Slenderness
  - Materials and section properties



Part 3: Members

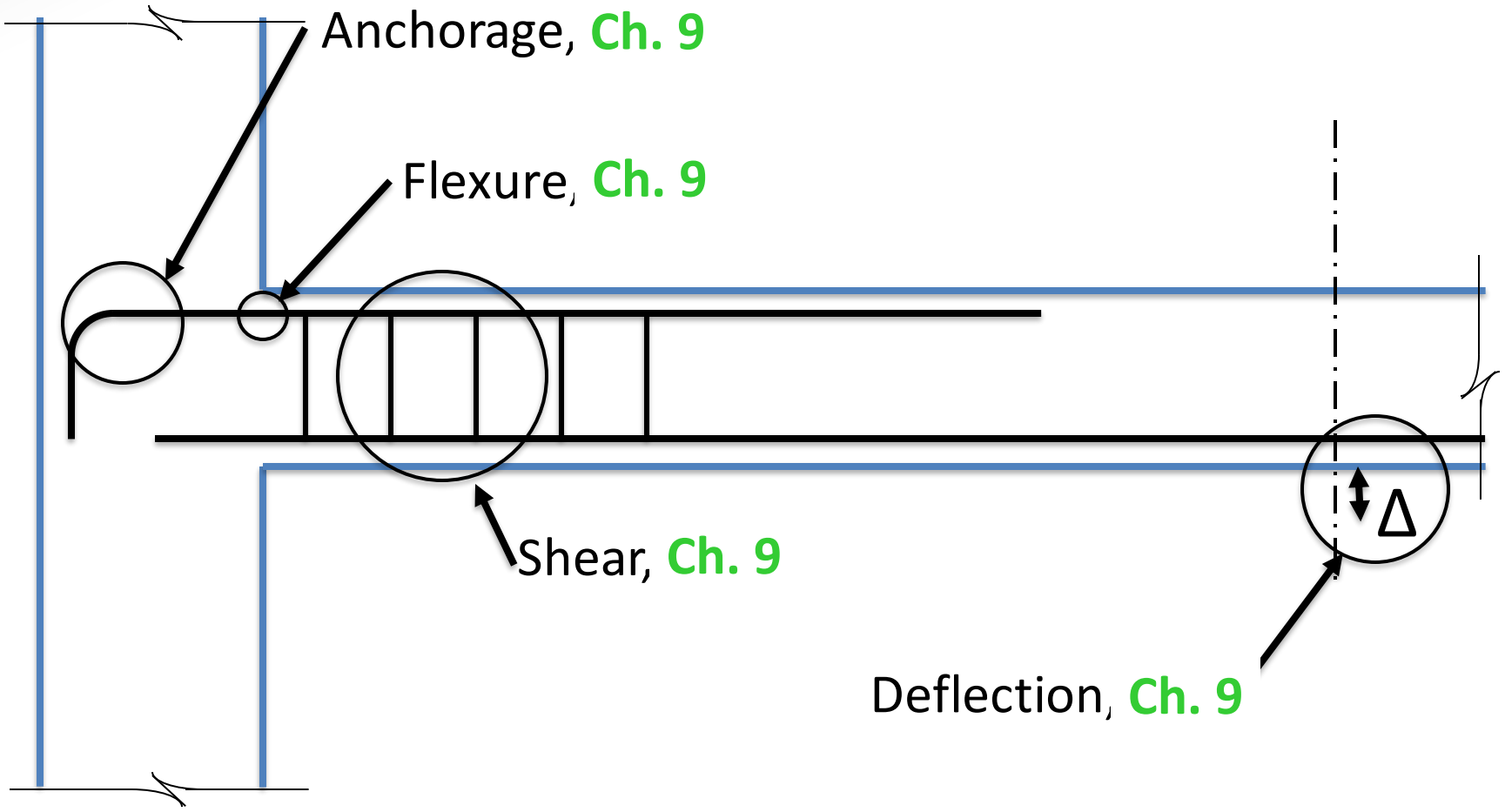
- 7: One-Way Slabs
- 8: Two-Way Slabs
- 9: Beams
- 10: Columns
- 11: Walls
- 12: Diaphragms
- 13: Foundations
- 14: Plain Concrete



Typical member chapter sections

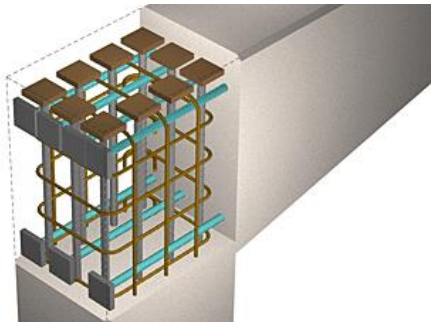
- X.1 Scope
- X.2 General
- X.3 Design Limits
- X.4 Required Strength
- X.5 Design Strength
- X.6 Reinforcement Limits
- X.7 Reinforcement Detailing
- X.? ?

ACI 318-19 Organization



Part 4: Joints / Connections / Anchors

- 15: Beam-column and slab-column joints



- 16: Connections between members



- 17: Anchoring to concrete



Part 5: Seismic

- 18: Earthquake Resistant Structures

Table R18.2—Sections of Chapter 18 to be satisfied in typical applications[1]

Component resisting earthquake effect, unless otherwise noted	SDC			
	A (None)	B (18.2.1.3)	C (18.2.1.4)	D, E, F (18.2.1.5)
Analysis and design requirements	None	18.2.2	18.2.2	18.2.2, 18.2.4
Materials		None	None	18.2.5 through 18.2.8
Frame members		18.3	18.4	18.6 through 18.9
Structural walls and coupling beams		None	None	18.10
Precast structural walls		None	18.5	18.5[2], 18.11
Diaphragms and trusses		None	18.12	18.12
Foundations		None	18.13	18.13
Frame members not designated as part of the seismic-force-resisting system		None	None	18.14
Anchors		None	18.2.3	18.2.3

[1]In addition to requirements of Chapters 1 through 17, 19 through 26, and ACI 318.2, except as modified by Chapter 18. Section 14.1.4 also applies in SDC D, E, and F.

[2]As permitted by the general building code.

Part 6: Materials & Durability

- 19: Concrete: Design and Durability Properties



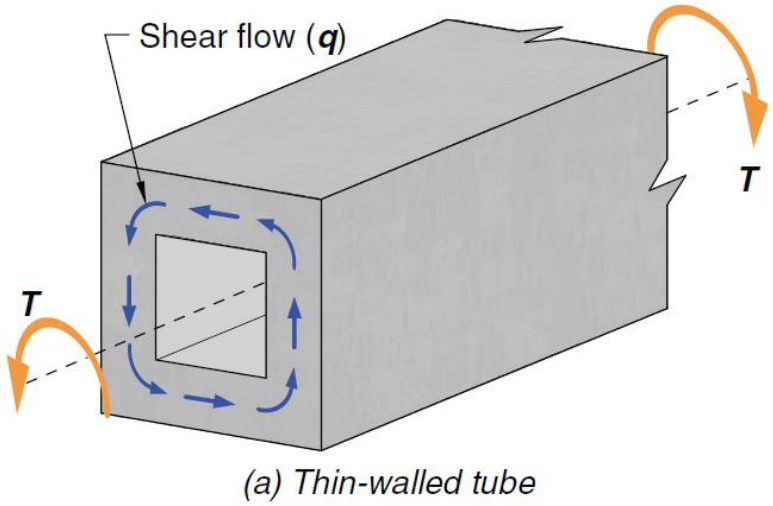
(Credit: PCA)

- 20: Steel Reinforcement Properties, Durability, and Embedments

Part 7: Strength & Serviceability

- 21: Strength Reduction Factors ϕ

- 22: Sectional Strength



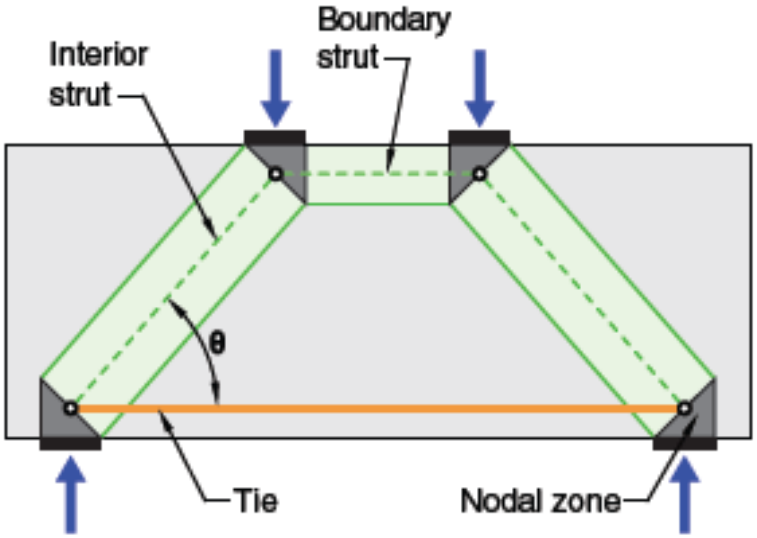
(a) Thin-walled tube

Organization

Member Chapter	Toolbox Chapter
9.5 — Design strength	
9.5.2 — Moment	
9.5.2.1 — If $P_u < 0.10f'_cA_g$, M_n shall be calculated in accordance with 22.3.	22.3 —Flexural strength… 22.3.3.4 …
9.5.2.2 — If $P_u \geq 0.10f'_cA_g$, M_n shall be calculated in accordance with 22.4.	22.4 — Axial strength or combined flexural and axial strength… 22.4.3.1 …

Part 7: Strength & Serviceability

- 23: Strut-and-Tie Method

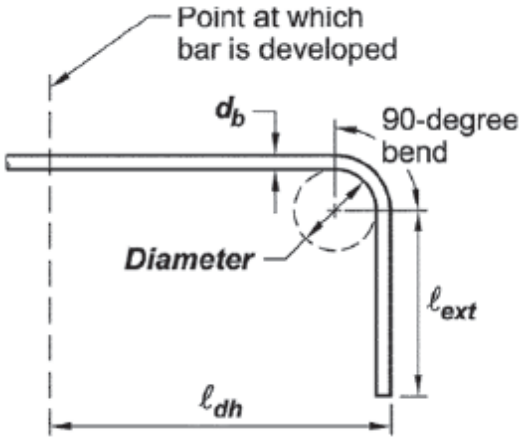


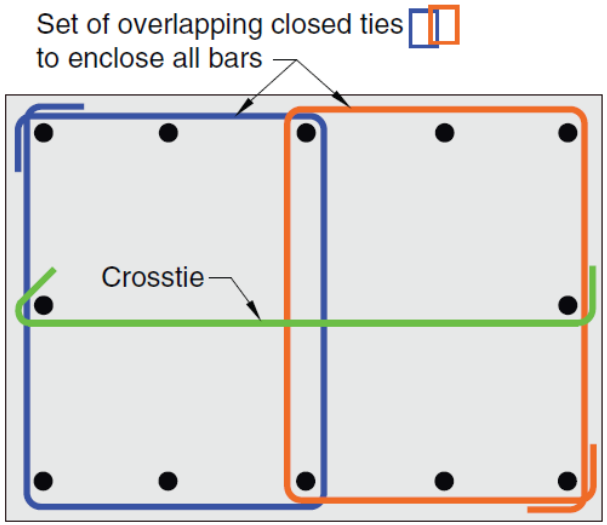
- 24: Serviceability

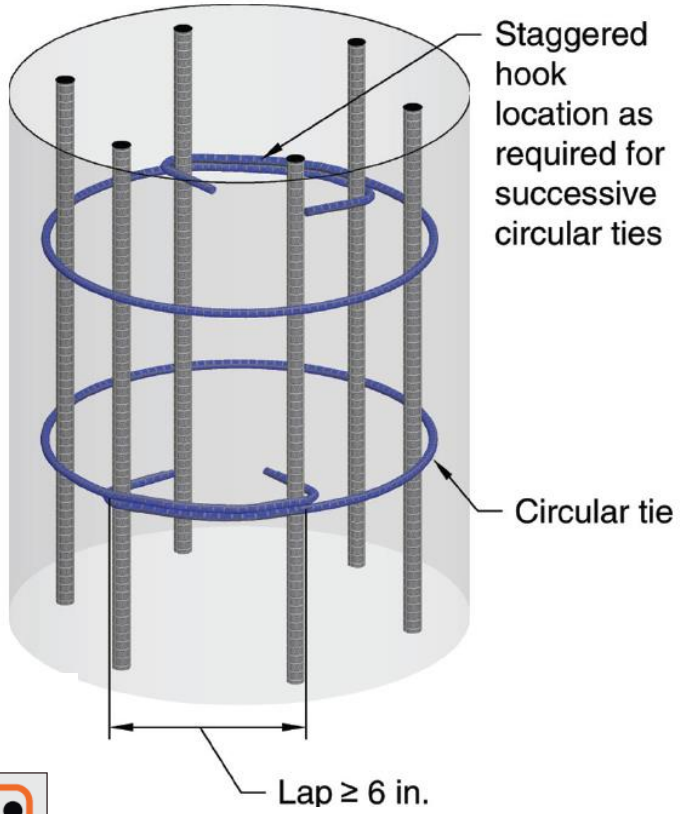
$$\Delta \leq l/360, I_{eff}$$

Part 8: Reinforcement

- 25: Reinforcement Details







Part 9: Construction

- **26: Construction Documents and Inspection**
  - 318 is written to the engineer, not the contractor.
  - Construction requirements must be communicated on the construction documents.
  - All construction requirements are gathered together in Chapter 26.
  - Design information – job specific
  - Compliance requirements – general quality
  - Inspection requirements



Part 10: Evaluation

- **27: Strength Evaluation of Existing Structures**
  - Applies when strength is in doubt
  - Well understood – analytical evaluation
  - Not well understood – load test



Benefits of ACI 318 organization

- Organized from a designer's perspective
- Easier to find specific requirements
- Intuitive location of information
- Clarified cross references
- Tables improve speed of understanding
- Consistent language in text
- Single idea for each requirement

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Existing Structures



1.4—Applicability

1.4.1 This Code shall apply to concrete structures designed and constructed under the requirements of the general building code.

…

1.4.3 Applicable provisions of this Code shall be permitted to be used for structures not governed by the general building code.

Concrete designs governed by other ACI codes



216 - Fire



307 - Chimneys



313 - Silos



332 – Residential



349 – Nuclear Facilities



350 – Environmental



359 – Nuclear Contain.



369 – Seismic Retrofit



376 – RLG Containment



437 – Strength Evaluation

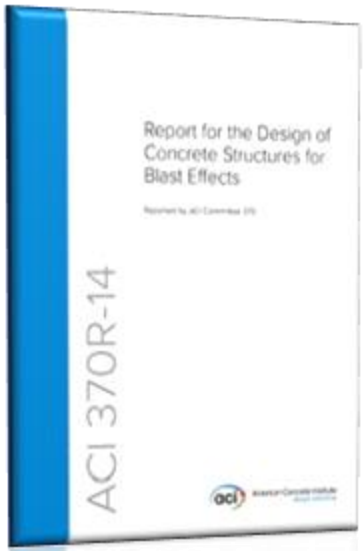


562 - Repair

Design recommendations provided in guides

- Slabs-on-ground (ACI 360R)
- Blast-resistant structures (ACI 370R)
- Wire Wrapped Tanks (ACI 372R)









1.4.2—Repair

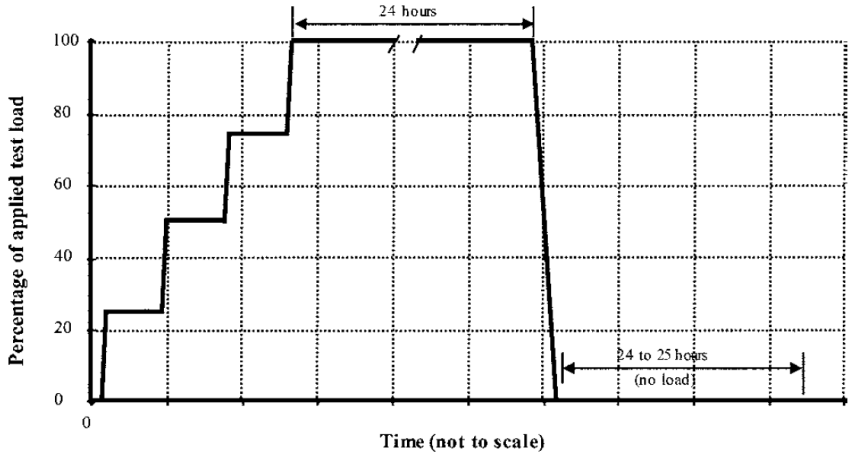
1.4.2 Provisions of this Code shall be permitted to be used for the assessment, repair, and rehabilitation of existing structures.

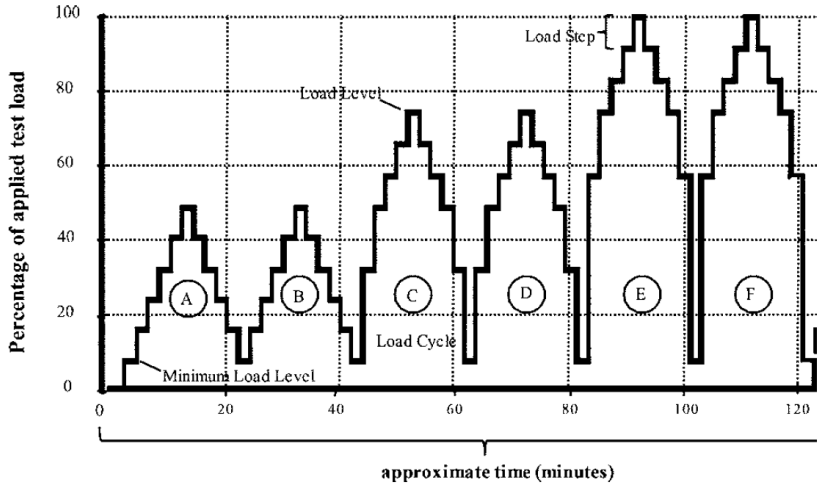
R1.4.2 Specific provisions for assessment, repair, and rehabilitation of existing concrete structures are provided in ACI 562-19. Existing structures in ACI 562 are defined as structures that are complete and permitted for use.

Chapter 27 – Strength Evaluation of Existing Structures

Applies when strength is in doubt

- **Well understood – analytical evaluation**
- **Not well understood – load test**
  - Monotonic procedure, ACI 318
  - Cyclic procedure, ACI 437.2





27.4.6.2—Total test load, T_t

Greatest of:

(a) $T_t = 1.15D + 1.5L + 0.4(L_r \text{ or } S \text{ or } R)$

$\rightarrow T_t = 1.0D_w + 1.1D_s + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$

(b) $T_t = 1.15D + 0.9L + 1.5(L_r \text{ or } S \text{ or } R)$

$\rightarrow T_t = 1.0D_w + 1.1D_s + 1.0L + 1.6(L_r \text{ or } S \text{ or } R)$

(c) $T_t = 1.3D$

$\rightarrow T_t = 1.3(D_w + D_s)$

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Loads & Analysis

Superposition of loads (R5.3.1)

- **Added commentary**
  - If the load effects such as internal forces and moments are linearly related to the loads, the required strength U may be expressed in terms of load effects with the identical result. If the load effects are nonlinearly related to the loads, such as frame P-delta effects (Rogowsky et al. 2010), the loads are factored prior to determining the load effects. Typical practice for foundation design is discussed in R13.2.6.1. Nonlinear finite element analysis using factored load cases is discussed in R6.9.3.

Superposition of loads (R5.3.1)

In other words:

- First order, linear analysis

  $M_{1.2D+1.6L} = 1.2\ M_D + 1.6\ M_L$

- Second order or nonlinear analysis

  $M_{1.2D+1.6L} \neq 1.2\ M_D + 1.6\ M_L$

Wind Loads (R5.3.5)

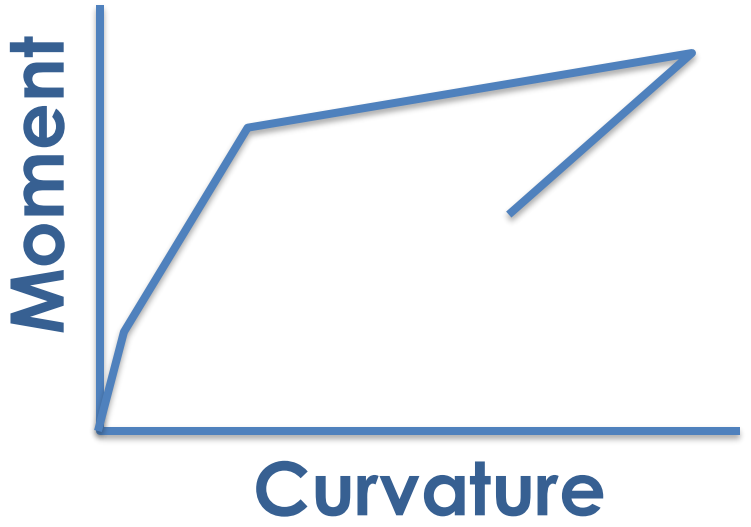
- **Added commentary**
  - ASCE 7-05
    - Wind = service-level wind
    - Use 1.6 load factor
  - ASCE 7-10 & ASCE 7-16
    - Wind = strength-level wind
    - Use 1.0 load factor





Inelastic First-Order Analysis (Chapter 6)

- **Not mentioned in ACI 318-14**
- **Nonlinear material properties**
- **Equilibrium satisfied in undeformed shape**
- **Several revisions**
  - Must consider column slenderness
  - No further redistribution
  - Clarifies requirements for each type of analysis

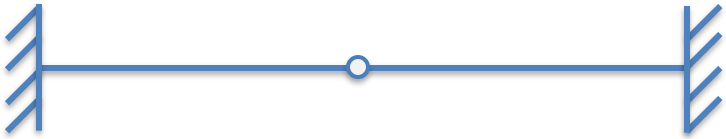


Consistent Stiffness Assumptions (6.3.1.1)

- ACI 318-14 dropped “consistent throughout the analysis” language



No top steel required



No bottom steel required



No steel required

Torsional Stiffness (R6.3.1.1)

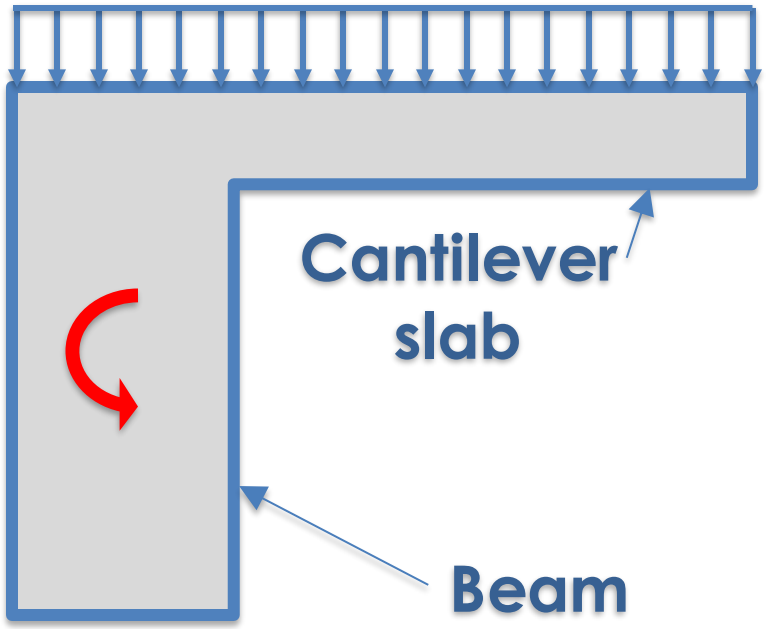
- **Clarification in commentary**
- **Two factors**
  - Torsional vs. flexural stiffnesses

$$GJ \quad \text{vs.} \quad EI$$

  - Equilibrium requirements

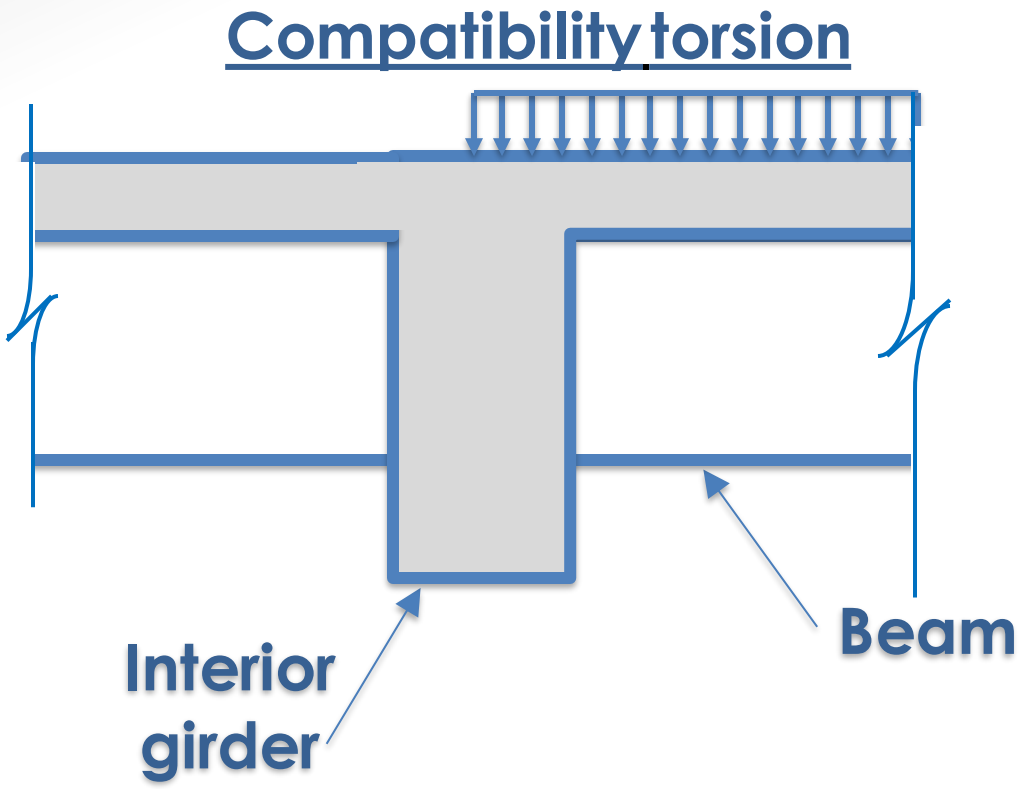
Torsional Stiffness

Equilibrium torsion



- Torsion in beam required to maintain equilibrium
- Torsion and torsional stiffness of the beam must be considered

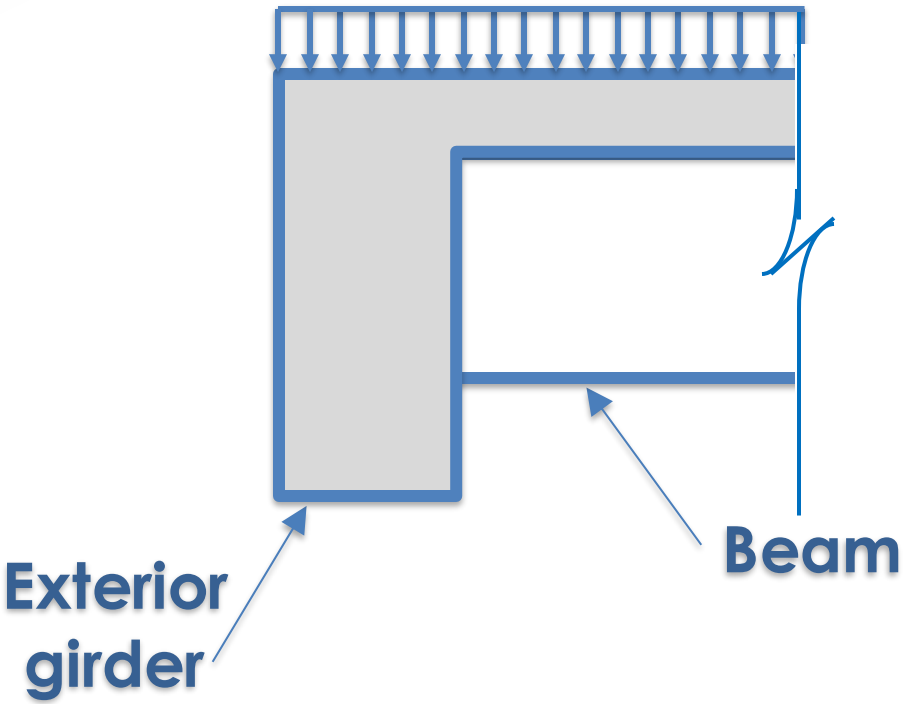
Torsional Stiffness



- Torsion in girder not required to maintain equilibrium
- Torsion and torsional stiffness of the beam may be neglected

Torsional Stiffness

Compatibility torsion



- Torsion in girder not required to maintain equilibrium
- Torsion and torsional stiffness of the girder should be included

Shear Area (6.6.3.1)

Table 6.6.3.1.1(a)— Moments of Inertia and cross-sectional areas permitted for elastic analysis at factored load level

Member and condition		Moment of inertia	Cross-sectional area for axial deformations	Cross-sectional area for shear deformations
Columns		$0.70I_g$	$1.0A_g$	$b_w h$
Walls	Uncracked	$0.70I_g$		
	Cracked	$0.35I_g$		
Beams		$0.35I_g$		
Flat plates and flat slabs		$0.25I_g$		

- **No previous guidance**



Floor Vibrations (R24.1)

- **Typical floors**
  - Good performance
- **Areas of concern**
  - Long/open spans
  - High-performance (precision machinery)
  - Rhythmic loading or vibrating machinery
  - Precast
- **Commentary references**

Floor Vibrations

- **Resources**
  - ATC Design Guide 1, "Minimizing Floor Vibration,"
  - Fanella, D.A., and Mota, M., "Design Guide for Vibrations of Reinforced Concrete Floor Systems,"
  - Wilford, M.R., and Young, P., "A Design Guide for Footfall Induced Vibration of Structures,"
  - PCI Design Handbook
  - Mast, R.F., "Vibration of Precast Prestressed Concrete Floors
  - West, J.S.; Innocenzi, M.J.; Ulloa, F.V.; and Poston, R.W., "Assessing Vibrations"

CIP

P-T Precast

- **No specific requirements**

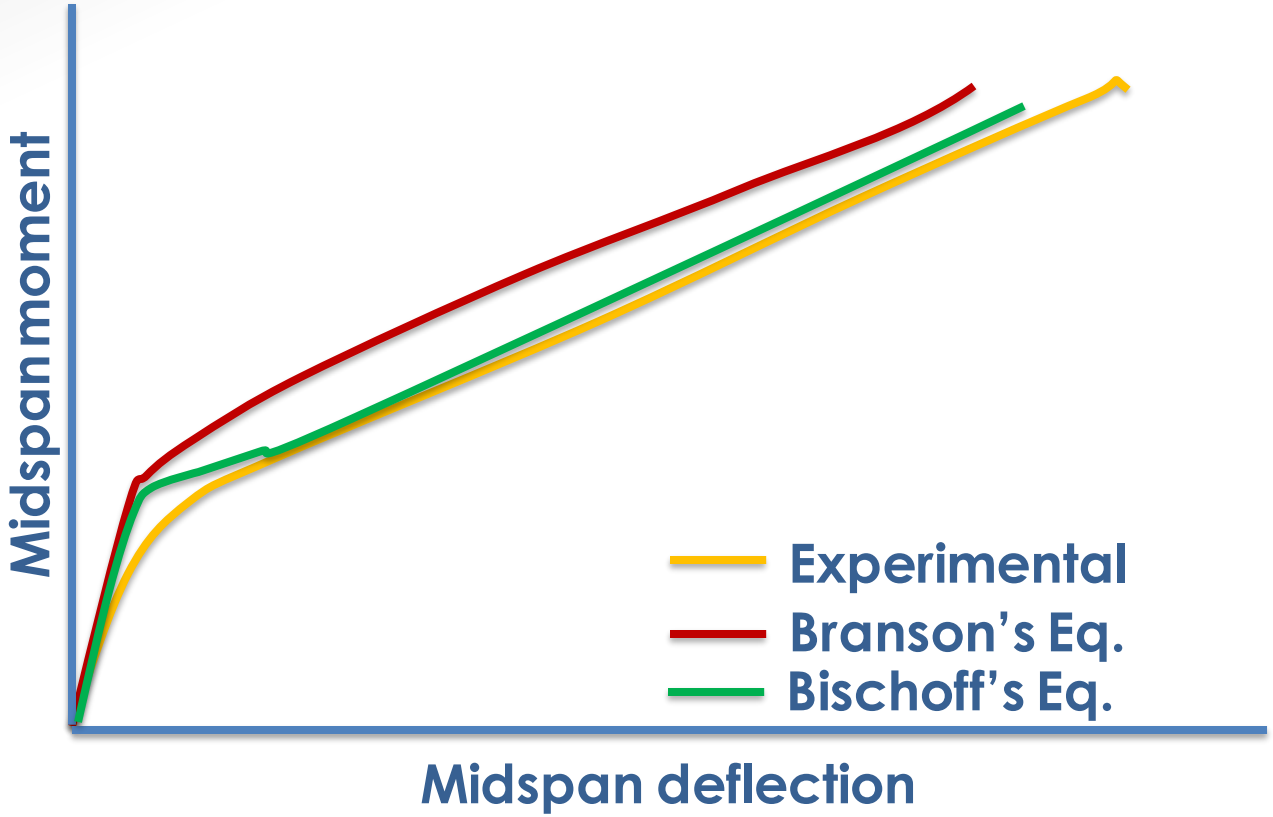
Concerns about deflection calculations

- Service level deflections based on Branson's equation underpredicted deflections for ρ below ≈ 0.8%

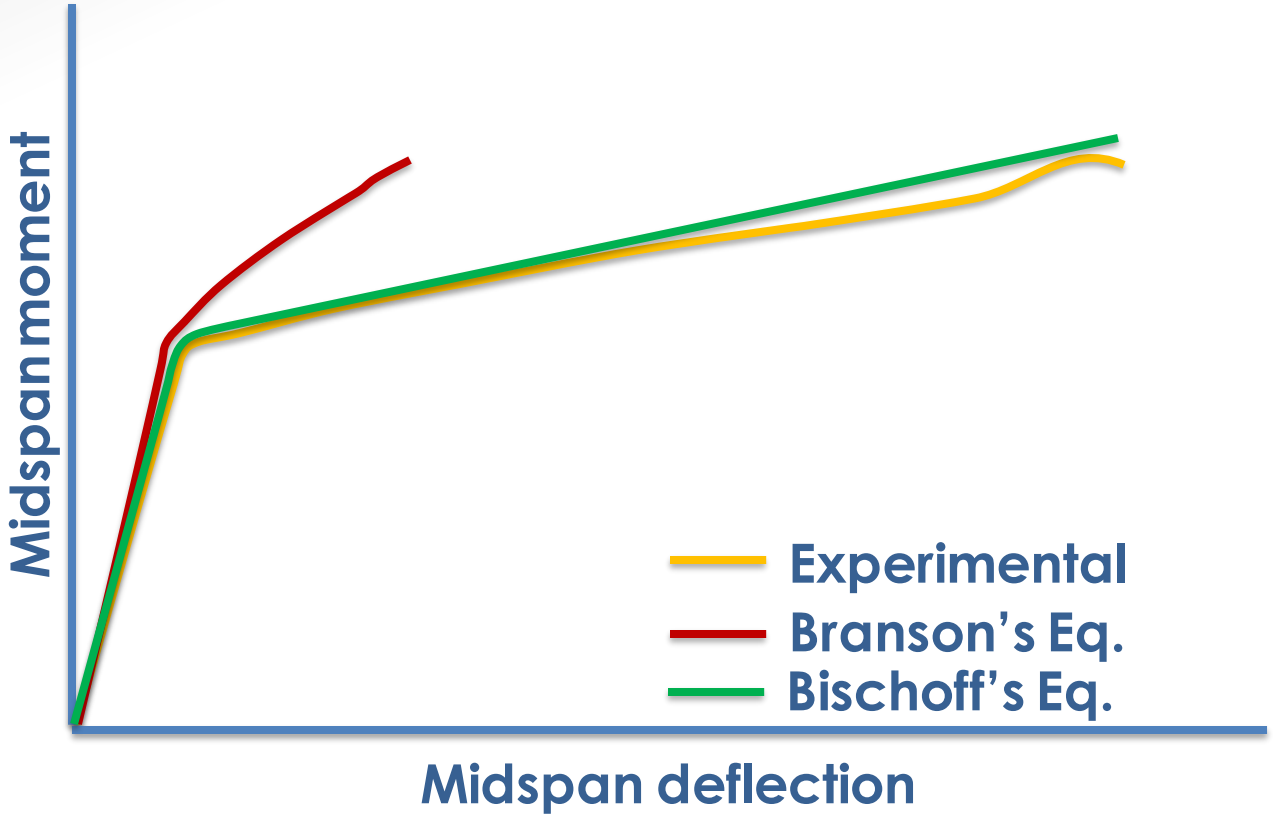
$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$

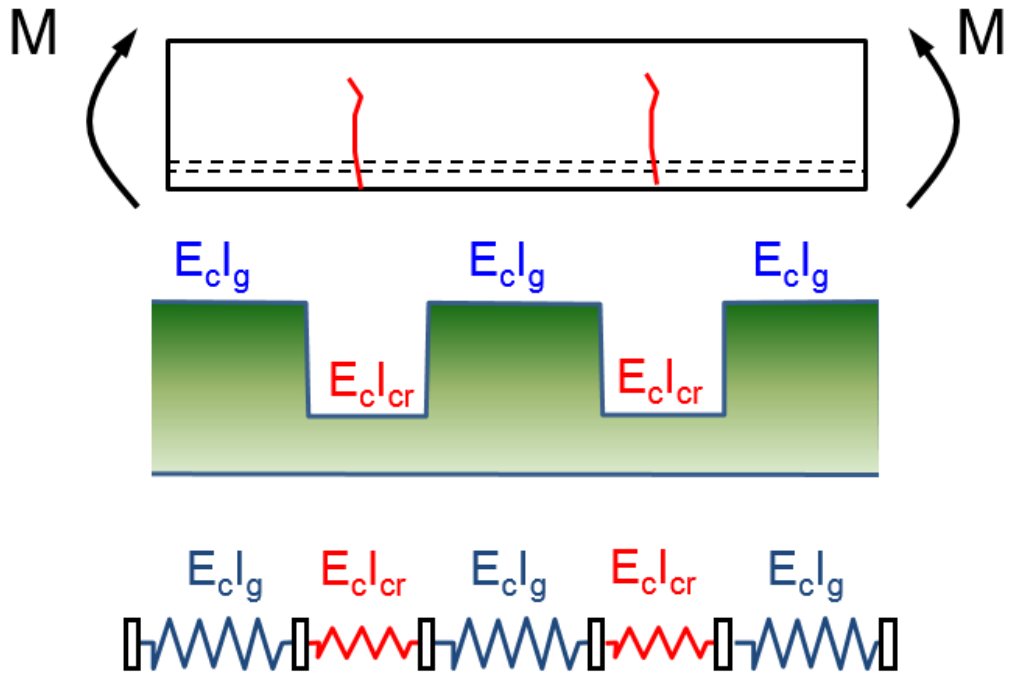
- Reports of excessive slab deflections (Kopczynski, Stivaros)
- High-strength reinforcement may result in lower reinforcement ratios

Heavily reinforced



Lightly reinforced

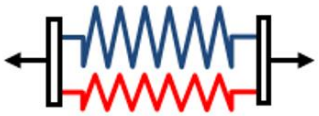




I_e should be the average of flexibilities

Comparison of Branson's and Bischoff's I_e

- **Branson**



$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^3\right) I_{cr} \leq I_g$$

- **Bischoff**



$$\frac{1}{I_e} = \left(\frac{M_{cr}}{M_a}\right)^2 \frac{1}{I_g} + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^2\right) \frac{1}{I_{cr}} \leq \frac{1}{I_g}$$

Branson combines stiffnesses. Bischoff combines flexibilities.

Effective Moment of Inertia

- **Table 24.2.3.5 ~ Inverse of Bischoff Eqn.**

$$M_a > (2/3)M_{cr}, I_e = \frac{I_{cr}}{1 - \left(\frac{(2/3)M_{cr}}{M_a}\right)^2 \left(1 - \frac{I_{cr}}{I_g}\right)}$$

$$M_a \leq (2/3)M_{cr}, I_e = I_g$$

- **2/3 factor added to account for:**
  - restraint that reduces effective cracking moment
  - reduced concrete tensile strength during construction
- ~~**Prestressed concrete**~~

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One-Way Slabs

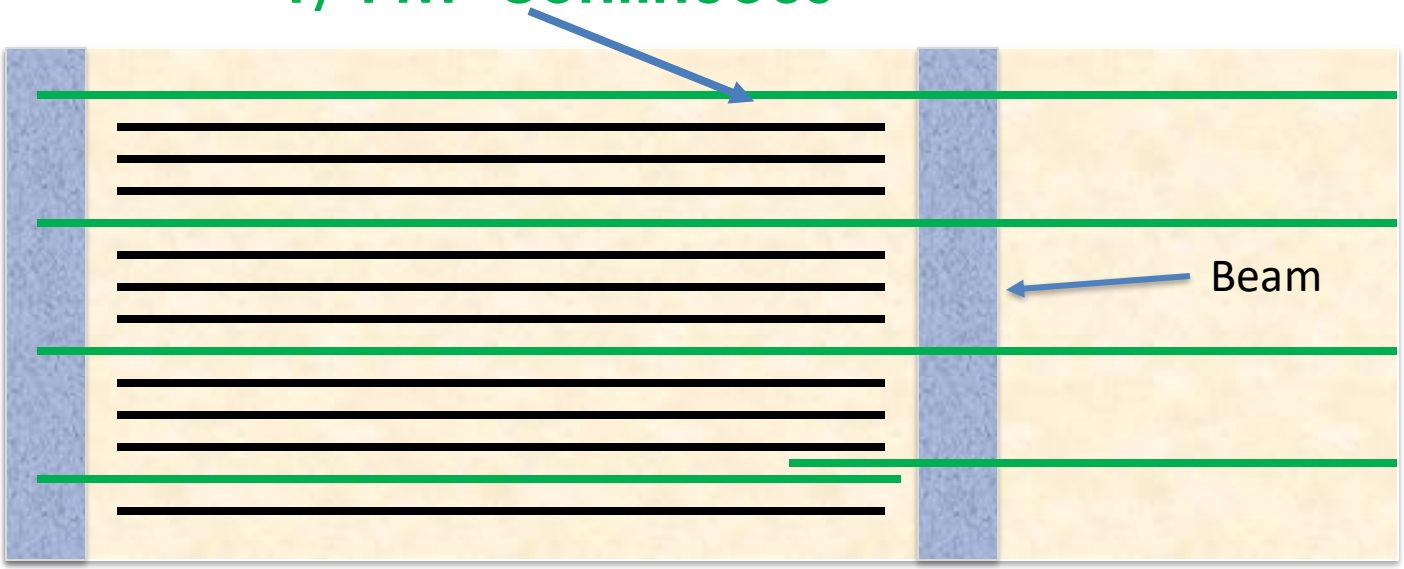
Structural Integrity Reinforcement

Structural integrity provisions have been added

- **To improve structural integrity**
  - To ensure that failure of a portion of a slab does not lead to disproportional collapse
- **To be similar to that for beams**
  - bring one-way cast-in-place slab structural integrity in line with beam structural integrity provisions

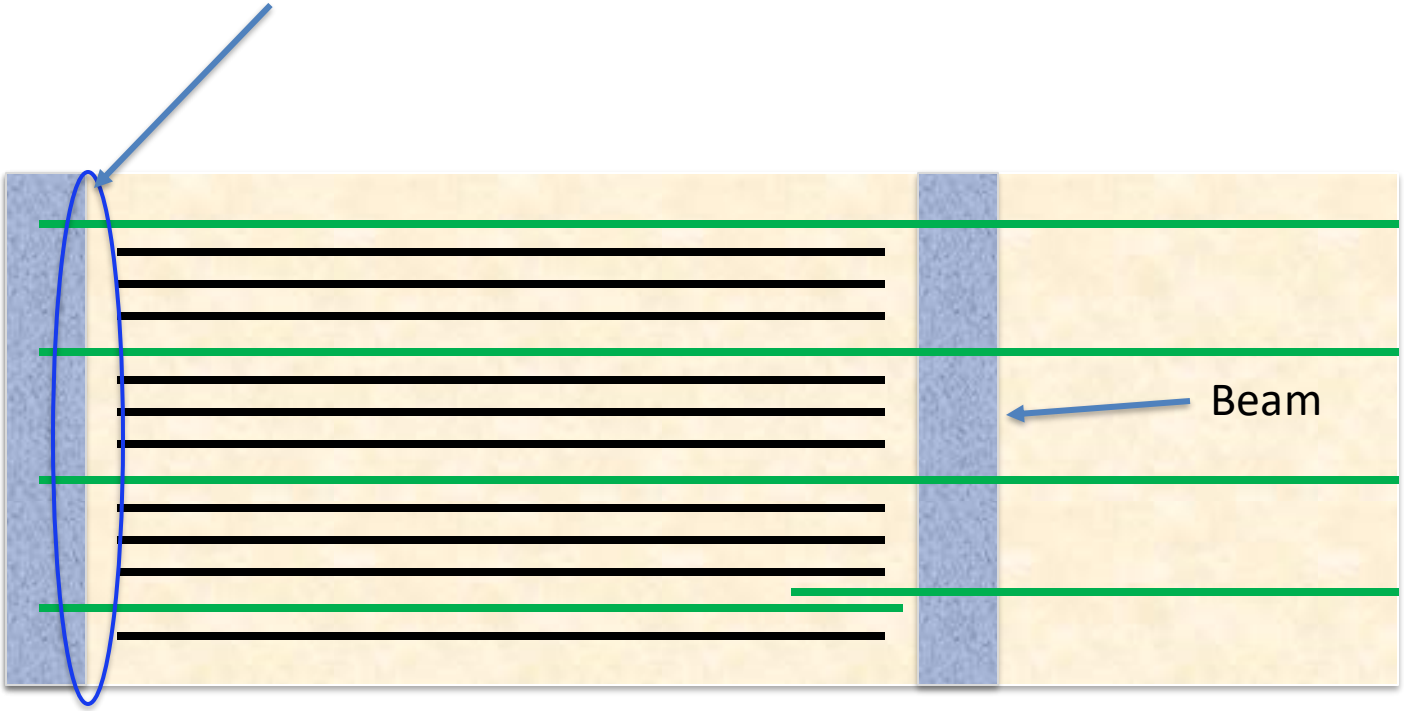
Structural Integrity Reinforcement

- **7.7.7 Structural integrity reinforcement in cast-in-place one-way slabs**
  - 7.7.7.1 Longitudinal reinf. consists of at least ¼ of max. positive moment to be continuous



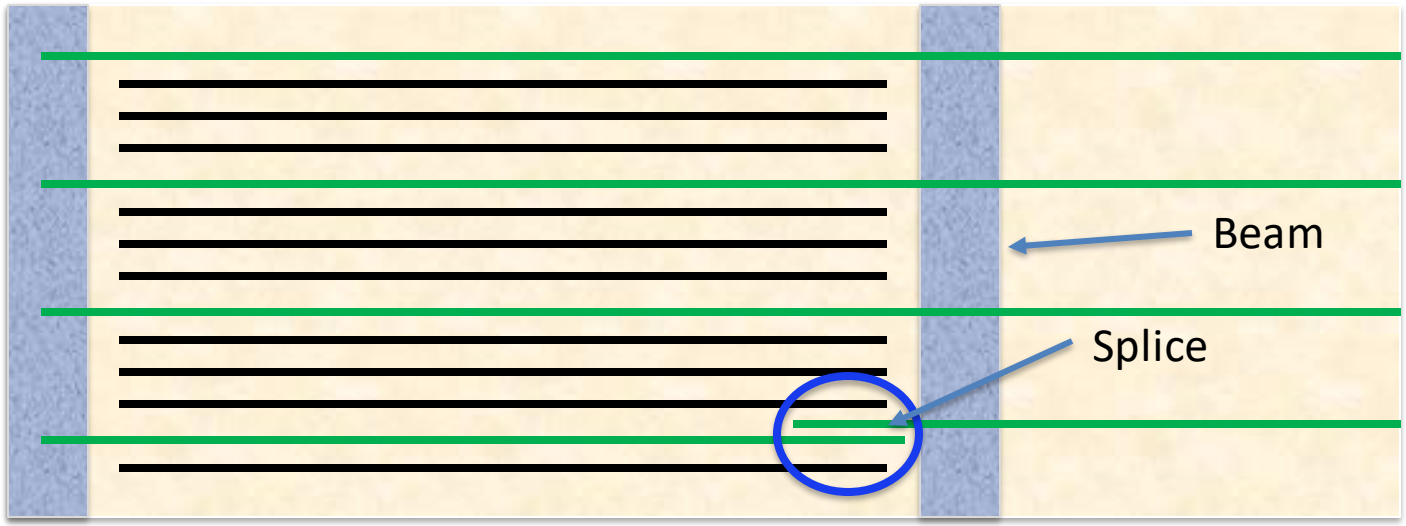
Structural Integrity Reinforcement

- 7.7.7.2 Longitudinal reinf. at noncontinuous supports to be **anchored** to develop f_y **at the face of the support**



Structural Integrity Reinforcement

- 7.7.7.3 Splices
  - Splice **near supports**
  - **mechanical or welded** in accordance with 25.5.2 or 25.5.7
  - **or Class B tension lap splices** in accordance with 25.5.2



Shrinkage and Temperature Reinforcement

7.6.4.1 → 24.4 Shrinkage and temperature reinforcement

24.4.3.2 : Ratio of deformed shrinkage and temperature reinforcement area to gross concrete area

- 318-14: as per **Table 24.4.3.2**

Table 24.4.3.2—Minimum ratios of deformed shrinkage and temperature reinforcement area to gross concrete area

Reinforcement type	f_y, psi	Minimum reinforcement ratio	
Deformed bars	< 60,000	0.0020	
Deformed bars or welded wire reinforcement	≥ 60,000	Greater of:	$\frac{0.0018 \times 60{,}000}{f_y}$
			0.0014

- 318-19: Ratio ≥ 0.0018

Minimum Flexural Reinforcement in Nonprestressed Slabs – One way

7.6.1.1:

- 318-14: $A_{s,min}$ as per Table 7.6.1.1

Table 7.6.1.1—$A_{s,min}$ for nonprestressed one-way slabs

Reinforcement type	f_y, psi	$A_{s,min}$	
Deformed bars	< 60,000	$0.0020A_g$	
Deformed bars or welded wire reinforcement	≥ 60,000	Greater of:	$\frac{0.0018 \times 60,000}{f_y} A_g$
			$0.0014A_g$

- 318-19: $A_{s,min} = 0.0018A_g$

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Two-Way Slabs

The Direct Design Method and The Equivalent Frame Method

- Removed: The direct design method (**8.10)** and the equivalent frame method (**8.11)**
- Provisions in 318-14
- **8.2.1** … The direct design method or the equivalent frame method is permitted.
- **6.2.4.1** Two-way slabs shall be permitted to be analyzed for gravity loads in accordance with (a) or (b):
  - (a) Direct design method for nonprestressed slabs
  - (b) Equivalent frame method for nonprestressed and prestressed slabs

Shearheads

- **Removed Shearhead provisions in 318-14**
  - 8.4.4.1.3 Slabs reinforced with shearheads shall be evaluated for two-way shear at critical sections in accordance with 22.6.9.8.

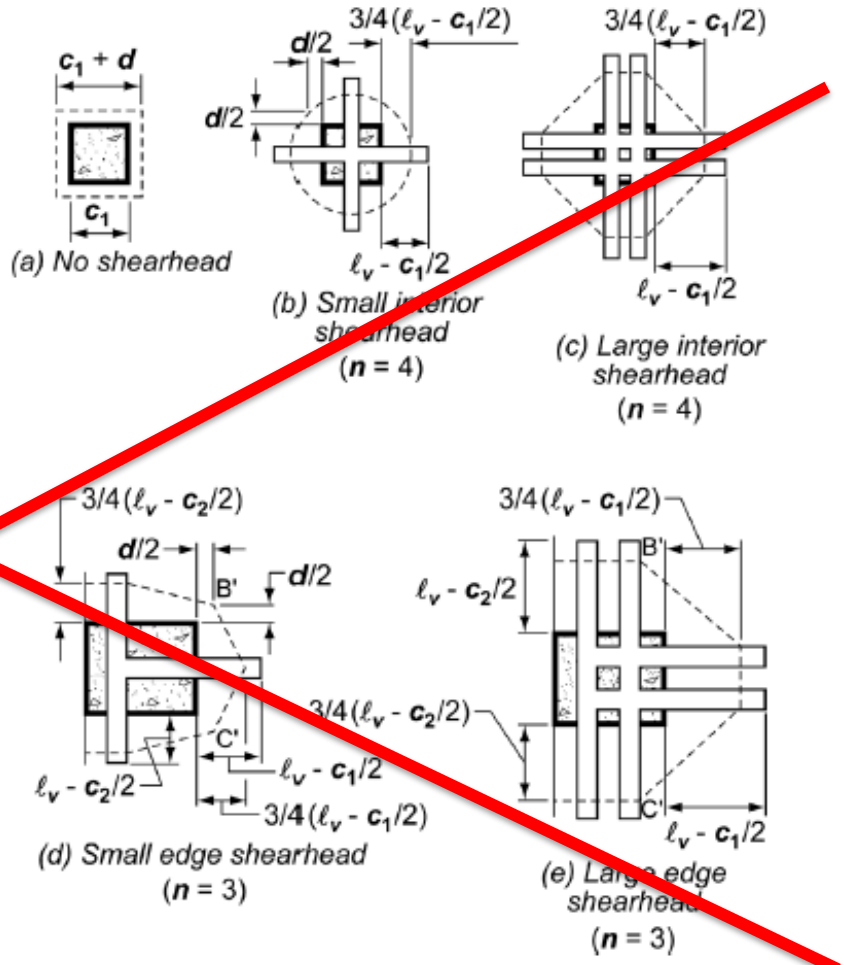


Fig. R22.6.9.8—Location of critical section defined in 22.6.9.8.

Opening in Slab Systems Without Beams

ACI 318 -14: 8.5.4.2(d)

- **within a column strip or closer than 10h from a concentrated load or reaction area satisfy**
  - 22.6.4.3 for slabs without shearheads
  - or 22.6.9.9 for slabs with shearheads

- **22.6.4.3: Reduced perimeter of critical section (b_o)**
  - Fig. R22.6.4.3

- **22.6.9.9: Reduction to b_o is ½ of that given in 22.6.4.3**

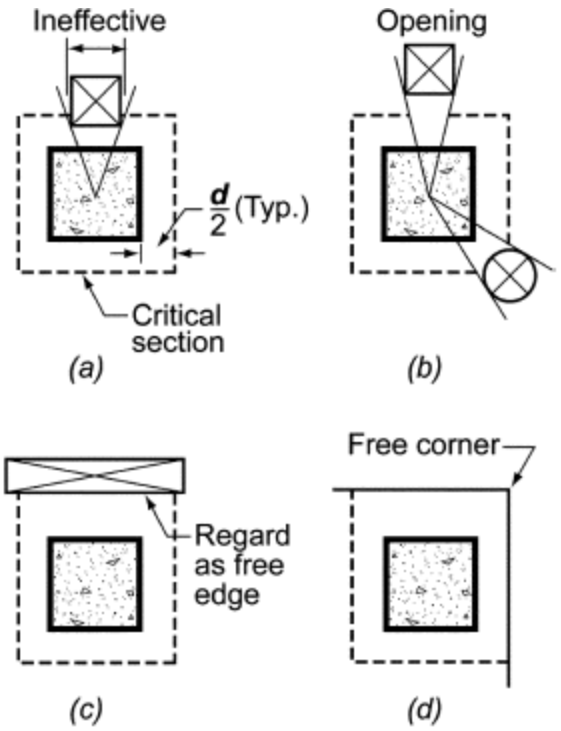


Fig. R22.6.4.3—*Effect of openings and free edges (effective perimeter shown with dashed lines)*

***Note**: Openings shown are located within **10h** of the column periphery*

Opening in Slab Systems Without Beams

ACI 318 -19: 8.5.4.2(d)

- **closer than 4h from the periphery of a column, concentrated load or reaction area satisfying 22.6.4.3**

- **22.6.4.3: Reduced perimeter of critical section (b_o)**
  - Fig. R22.6.4.3

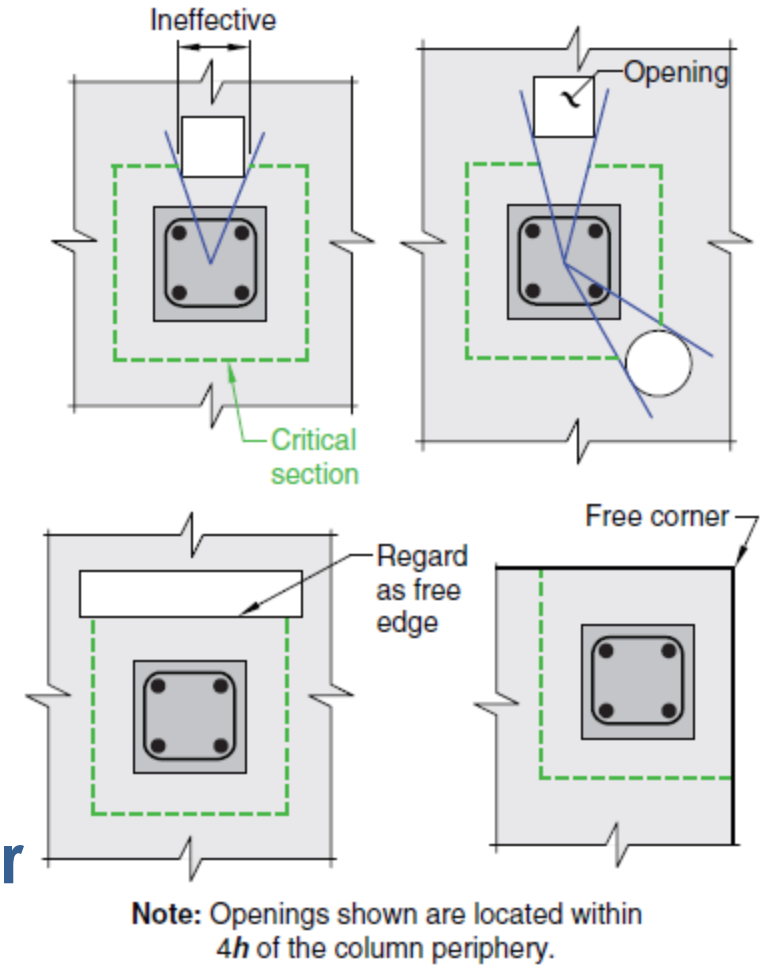


Fig. R22.6.4.3—*Effect of openings and free edges (effective perimeter shown with dashed lines).*

Minimum Flexural Reinforcement in Nonprestressed Slabs – Two way

8.6.1.1

- 318-14 : $A_{s,min}$ as per Table 8.6.1.1.

Table 8.6.1.1—$A_{s,min}$ for nonprestressed two-way slabs

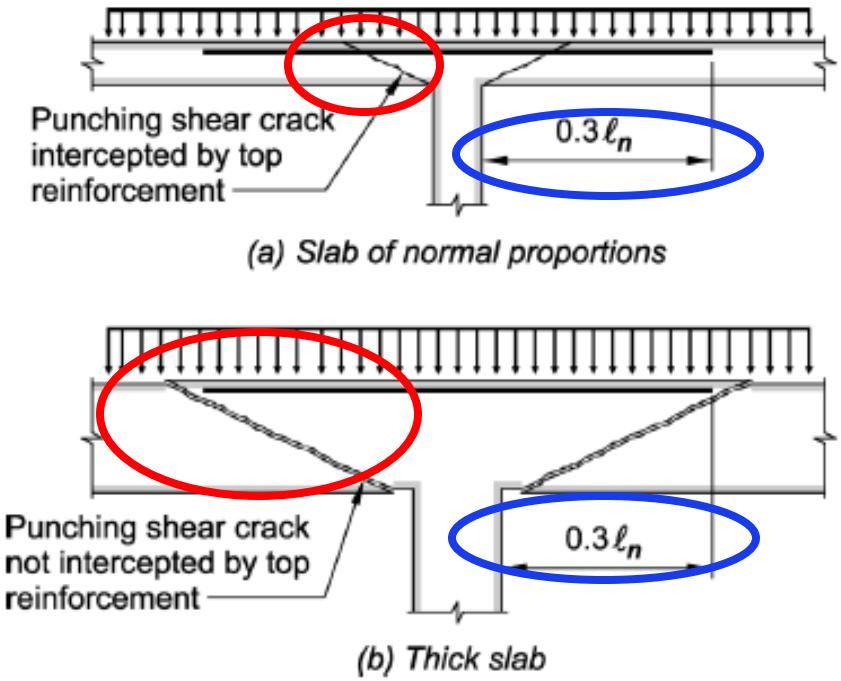
Reinforcement type	f_y, psi	$A_{s,min}$, in.^{2}	
Deformed bars	< 60,000	$0.0020A_g$	
Deformed bars or welded wire reinforcement	≥ 60,000	Greater of:	$\frac{0.0018 \times 60,000}{f_y} A_g$
			$0.0014A_g$

- 318-19: $A_{s,min}$ of $0.0018A_g$, or as defined in 8.6.1.2 (discussed under two-way shear)

Reinforcement Extensions for Slabs without Beams

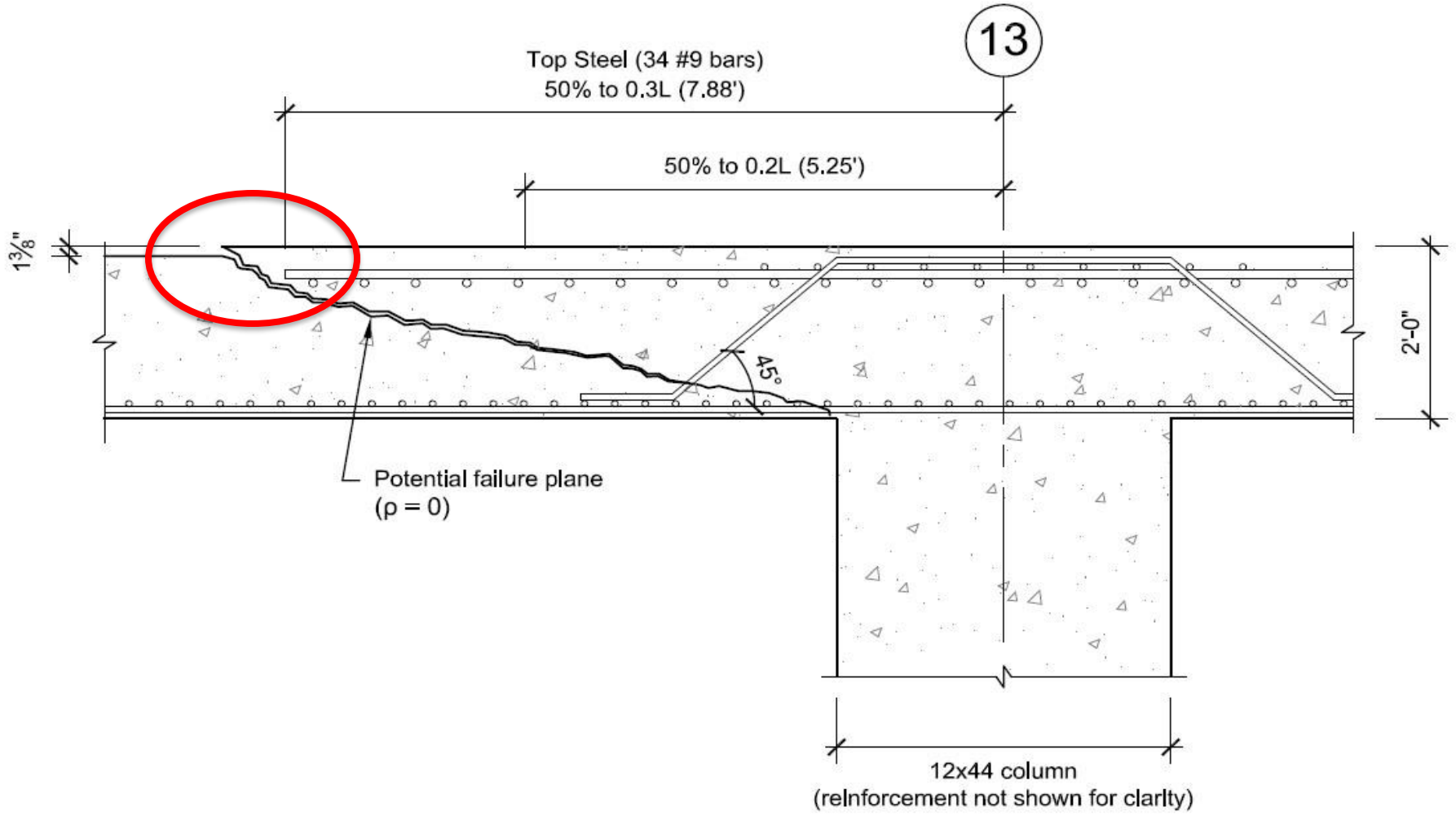
ACI 318-14: 8.7.4.1.3 - Column strip top bars

- **Extend to at least $0.3\ell_n$**
- **May not be sufficient for thick slabs**
  - may not intercept critical punching shear crack
  - Reduce punching shear strength



Punching shear cracks in slabs with reinforcement extensions

Punching shear failure - Podium Slab



- The failure crack did not intercept the top reinforcement.

Reinforcement Extensions for Two-Way Slabs without Beams

ACI 318-19: 8.7.4.1.3 - Column strip top bars

- Extend to at least $0.3\ell_n$ but, not less than 5d

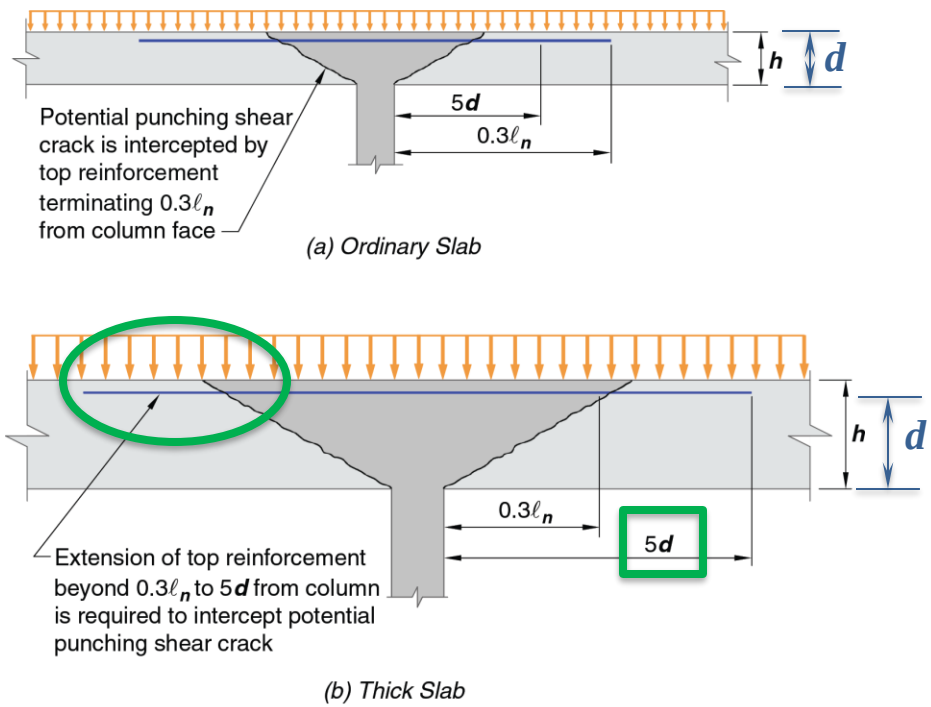


Fig. R8.7.4.1.3 - Punching shear cracks in ordinary and thick slabs

Reinforcement Extensions for Two-Way Slabs without Beams

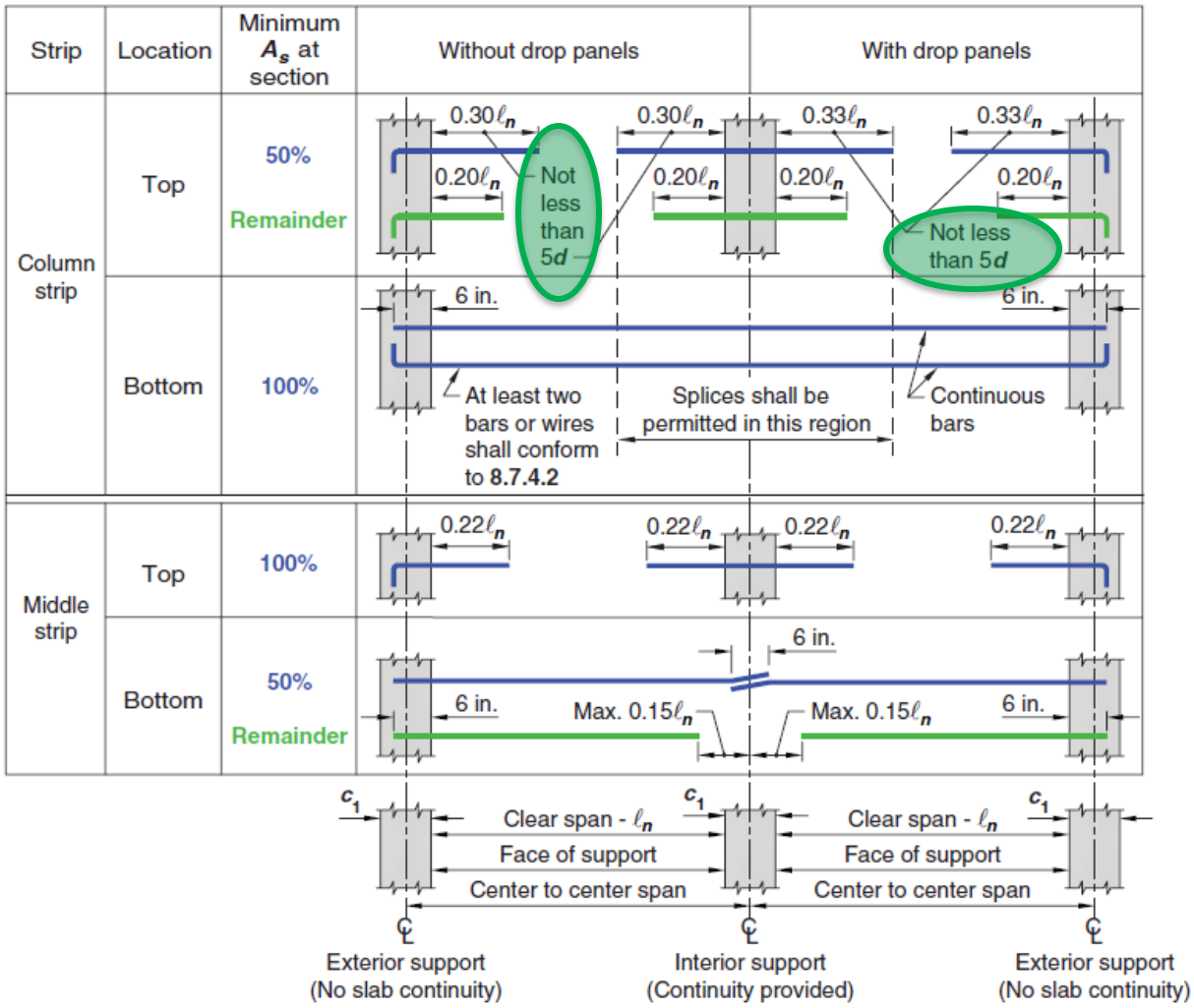


Fig. 8.7.4.1.3—Minimum extensions for deformed reinforcement in two-way slabs without beams.

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Post-tensioning

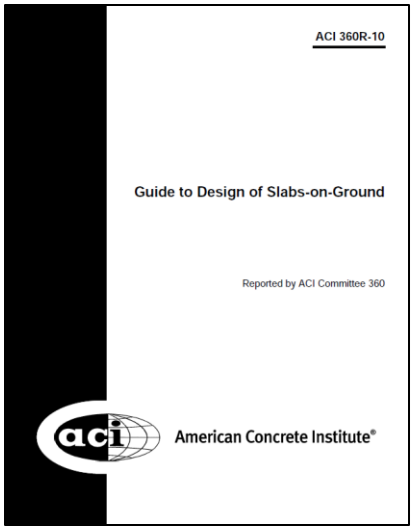
Residential P-T Slabs (1.4.6)

- **Past confusion about P-T slab foundation design on expansive soils**
  - Intent was for residential, but not mentioned with residential design provisions
- **Commentary clarifies use of PTI DC10.5-12 for P-T residential slabs and foundations on expansive soils**

Residential P-T Slabs (1.4.6)

- Coordinates with 2015 IBC requirements
- Adds reference to ACI 360 if not on expansive soil





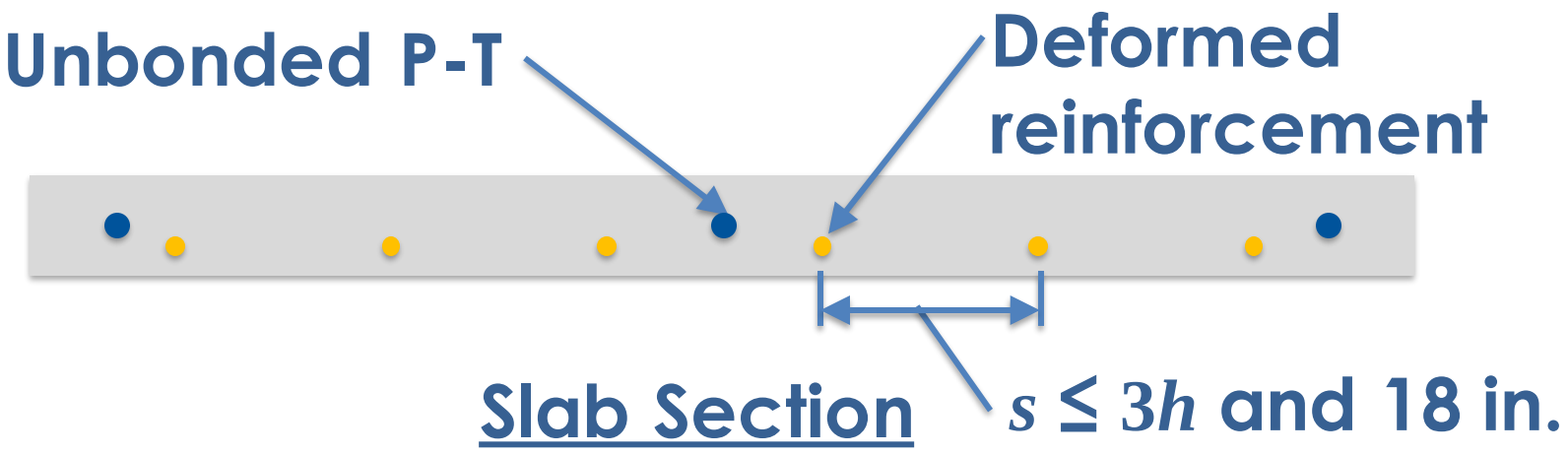


Max. Spacing of Deformed Reinf. (7.7.2.3)

- Class C (Cracked) and T (Transition) one-way slabs with unbonded tendons rely on bonded reinforcement for crack control
- Previously no limits for spacing of deformed reinforcement for Class C and T prestressed slabs
- Industry feedback provided

Max. Spacing of Deformed Reinf. (7.7.2.3)

- New limit is $s \leq 3h$ and 18 in.
- Same as non-prestressed slabs

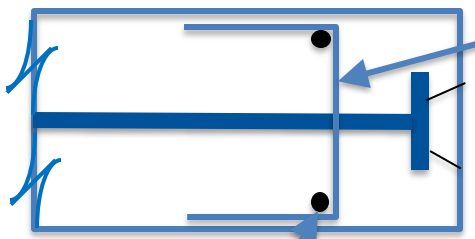


P-T Anchorage Zone Reinforcement (25.9.4.4.6)

- Referenced from slab and beam chapters
- Applies for groups of 6 or more anchors in thick slabs
- Anchorage zone requires backup bars for bearing and hairpins for bursting
- Hairpins must be anchored at the corners



Backup bars



Hairpins

Anchor bars



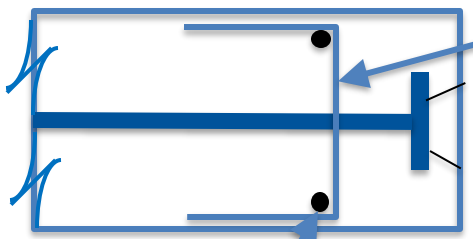
P-T Anchorage Zone Reinforcement (25.9.4.4.6)

- Thin slabs ≤ 8 in. → Anchor bars serve as backup bars

- Thick slabs > 8 in. → Both backup bars and anchor bars required



Backup bars



Hairpins

Anchor bars

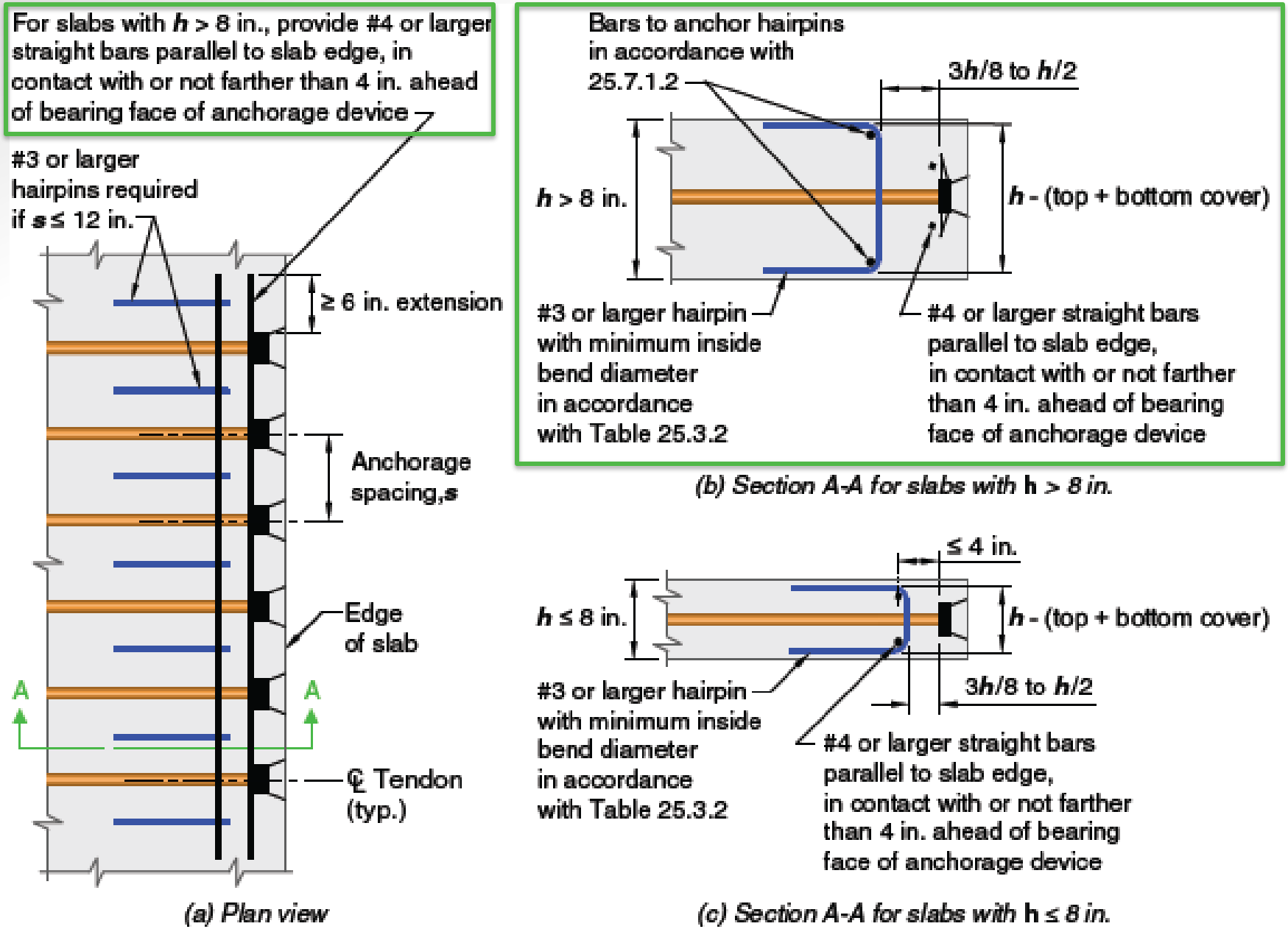
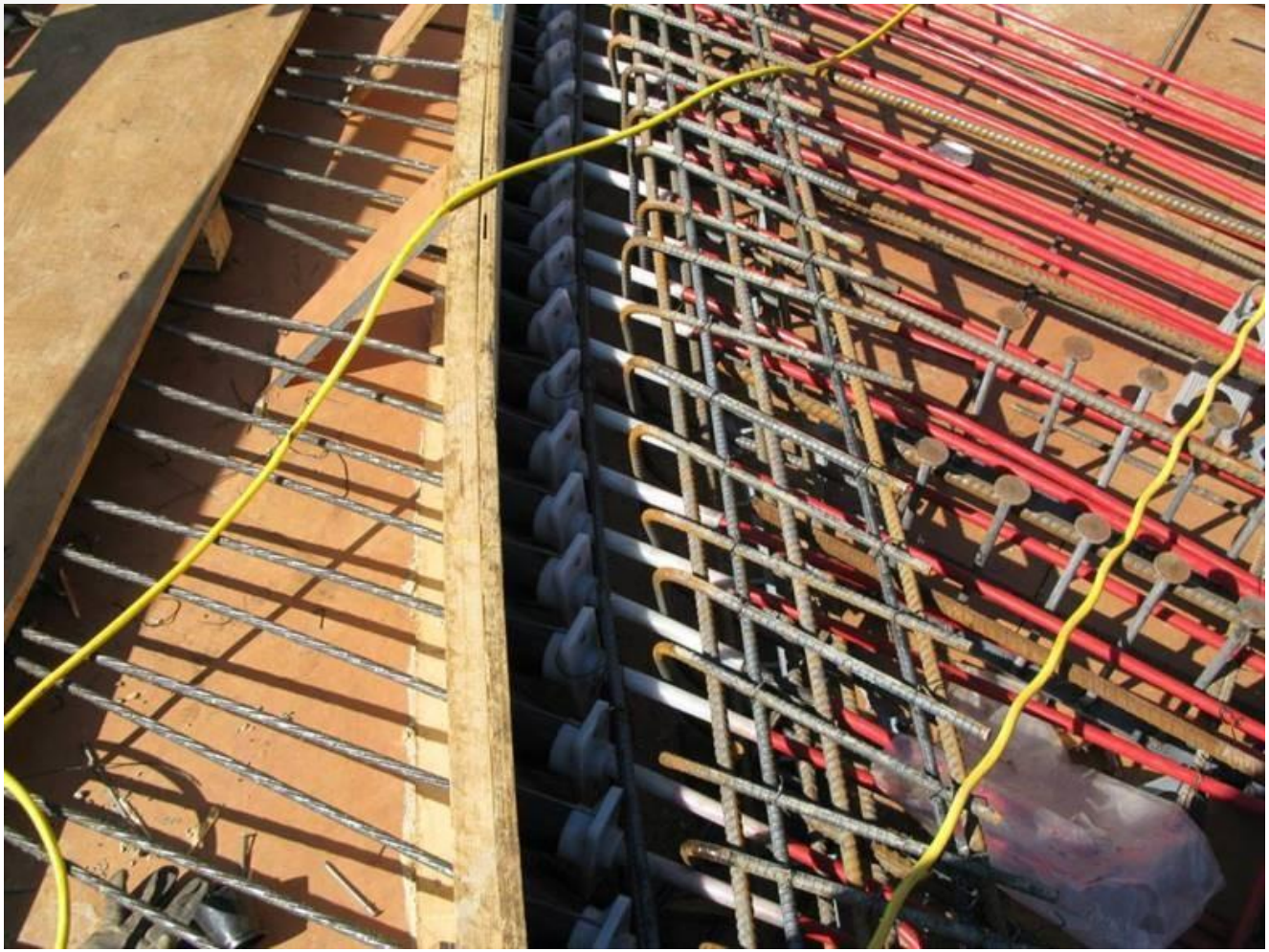


Fig. R25.9.4.4.6—Anchorage zone reinforcement for groups of 1/2 in. or smaller diameter tendons in slabs (other reinforcement not shown).





Design of Formwork for P-T (26.11.1.2 (5) and (6))

- Members may move when P-T strand is stressed
- Movement may redistribute loads
- Added requirement to allow for movement during tensioning
- Added requirement to consider redistribution of loads on formwork from tensioning of the prestressing reinforcement

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Precast/Prestressed

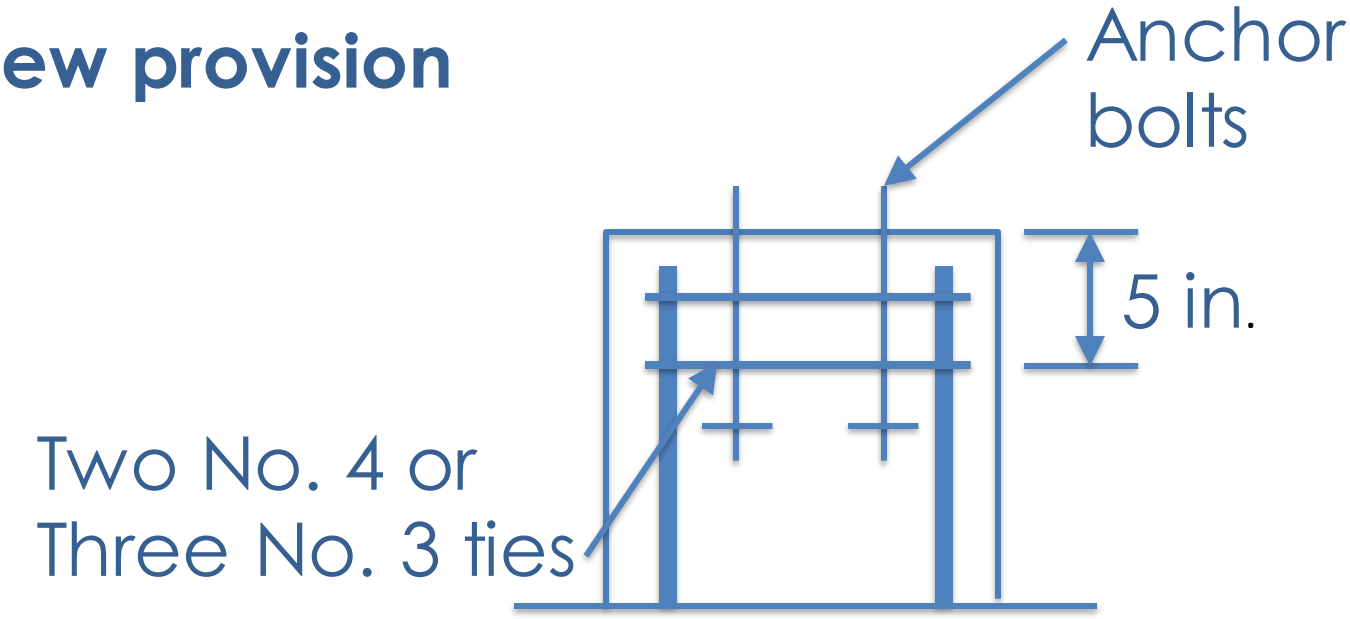
Precast/Prestressed Concrete

- Confinement for column/pedestal tops
- Connection forces
- Construction document requirement
- ϕ at ends of precast members



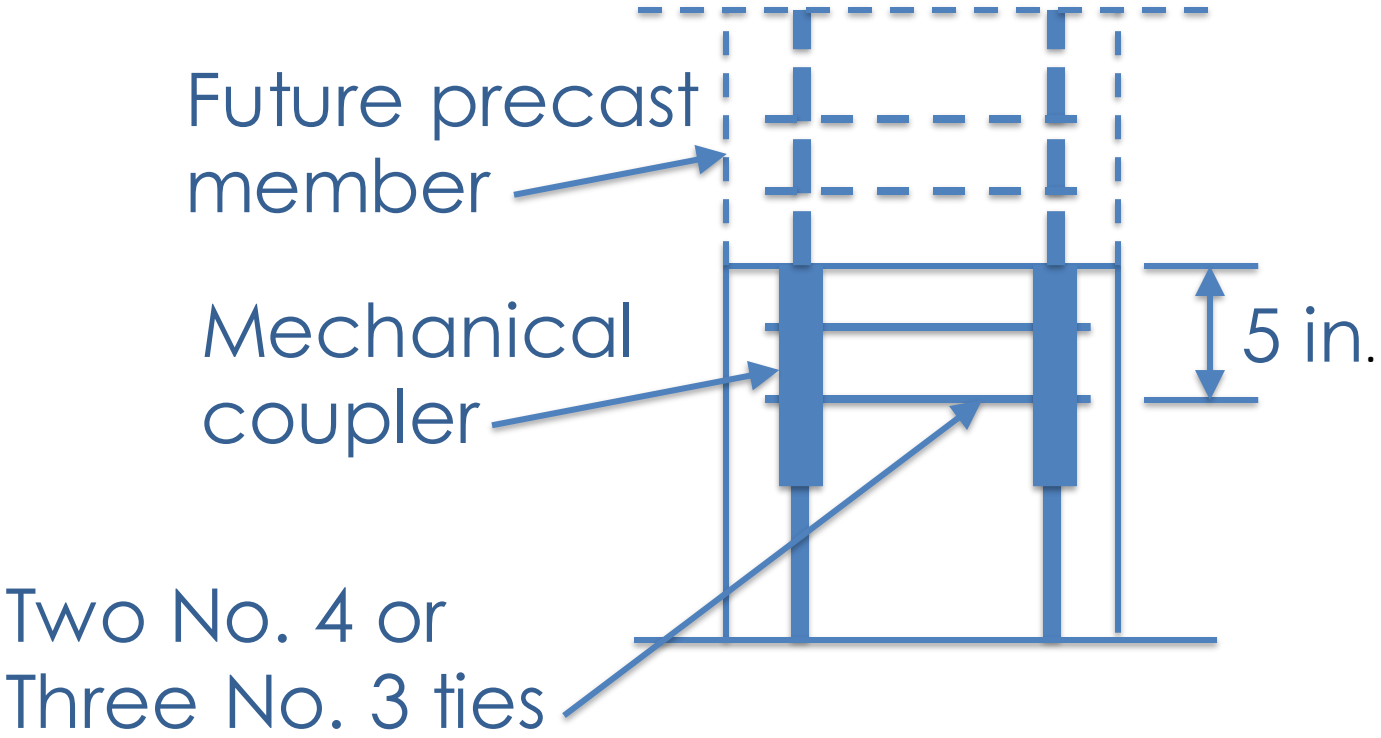
Confinement

- **10.7.6.1.5: confinement required at tops of columns/pedestals**
- **Assists in load transfer**
- **Not a new provision**



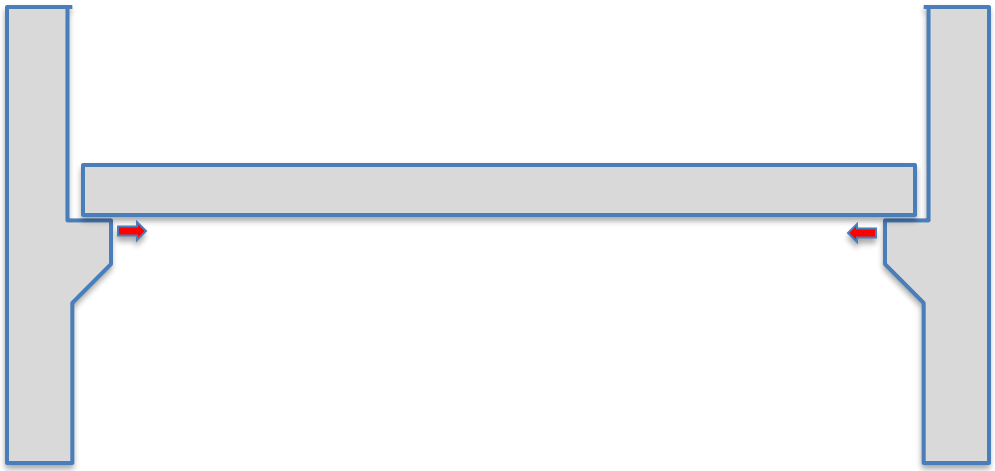
Confinement

- **10.7.6.1.6: extends confinement requirement to precast columns/pedestals**



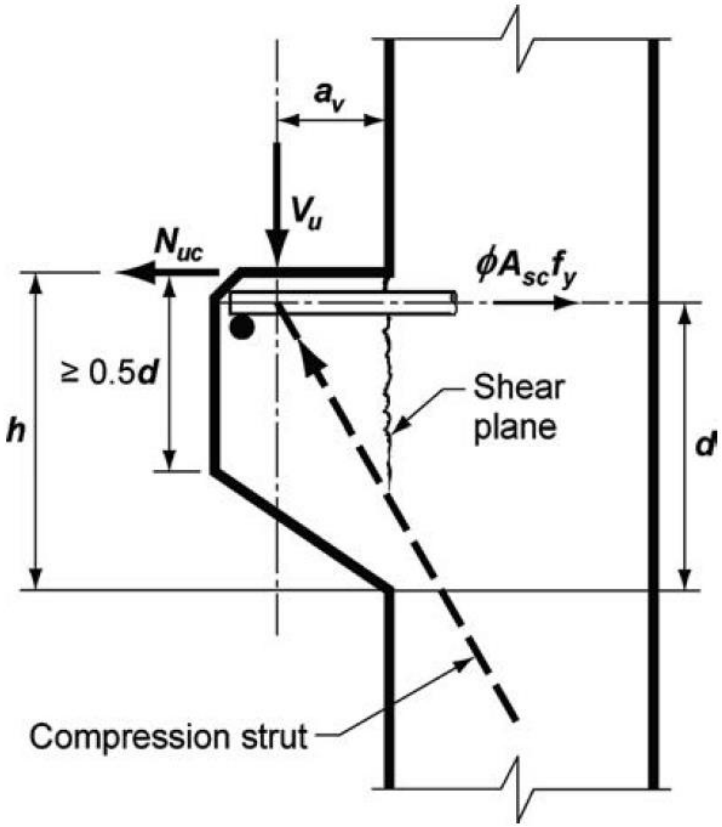
Volume Change in Precast Connections

- **Volume change**
  - Creep
  - Shrinkage
  - Temperature
- **May induce connection reactions if restrained**



Volume Change in Precast Connections

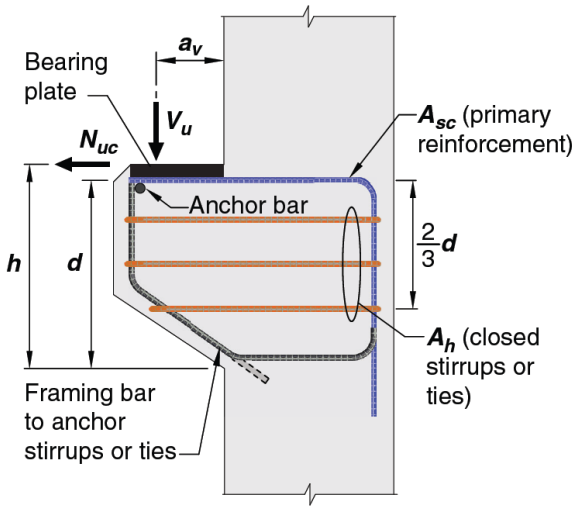
- **Load magnitude?**
- **Load factor?**
- **Past guidance for brackets and corbels**
  - Use $N_{UC} \geq 0.2V_U$ as restraint force
  - Use a 1.6 load factor
- **Approach was often to design around forces**



Volume Change and Connections

318-19 changes (16.2.2.3)

- N_{uc} = factored restraint force, shall be (a) or (b)
  - (a) restraint force x LL factor (no bearing pad)
  - (b) 1.6 x 0.2(sustained unfactored vertical load) for connections on bearing pads
- $N_{uc,max} \leq$ connection capacity x LL factor
- $N_{uc,max} \leq 1.6 \times \mu \times$ (sustained unfactored vertical load) if μ is known, (See 16.2.2.4)



Brackets and Corbels

- **26.6.4.1(a)** Details for welding of anchor bars at the front face of brackets or corbels designed by the licensed design professional in accordance with 16.5.6.3(a).

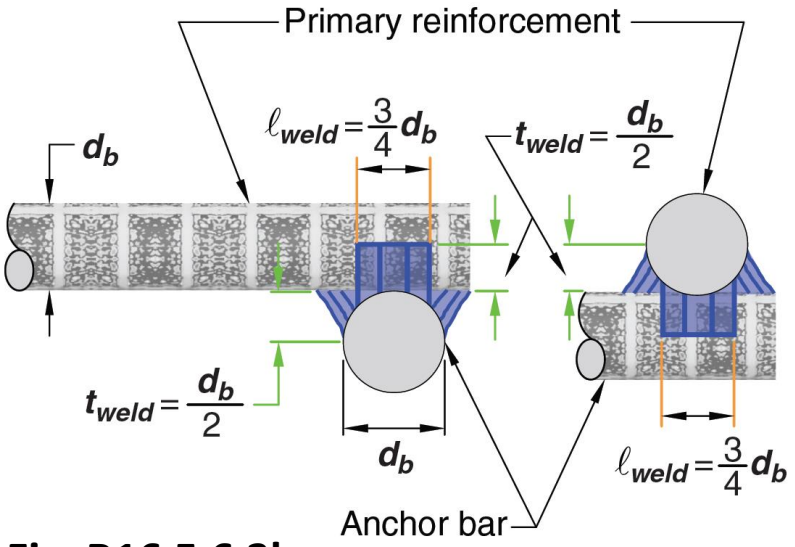


Fig. R16.5.6.3b

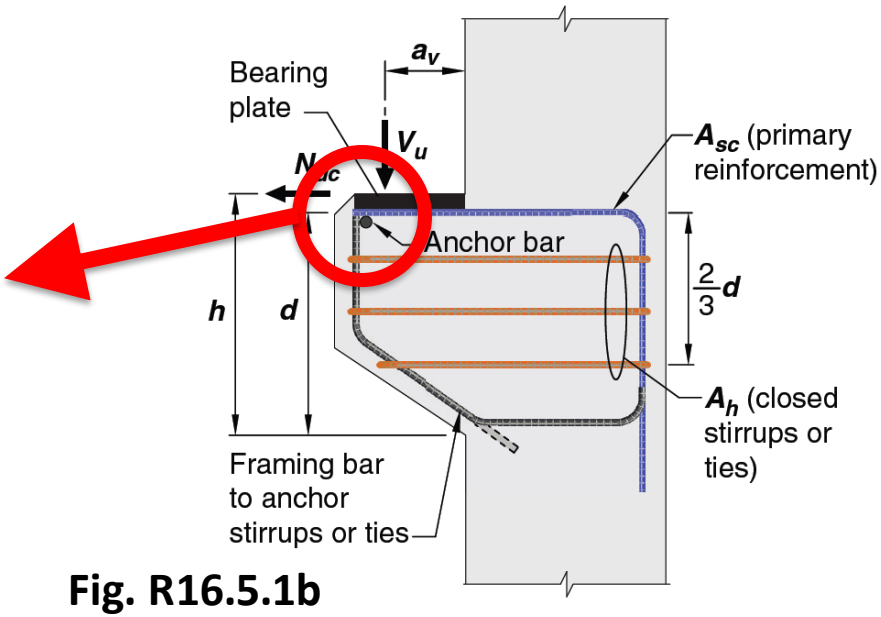
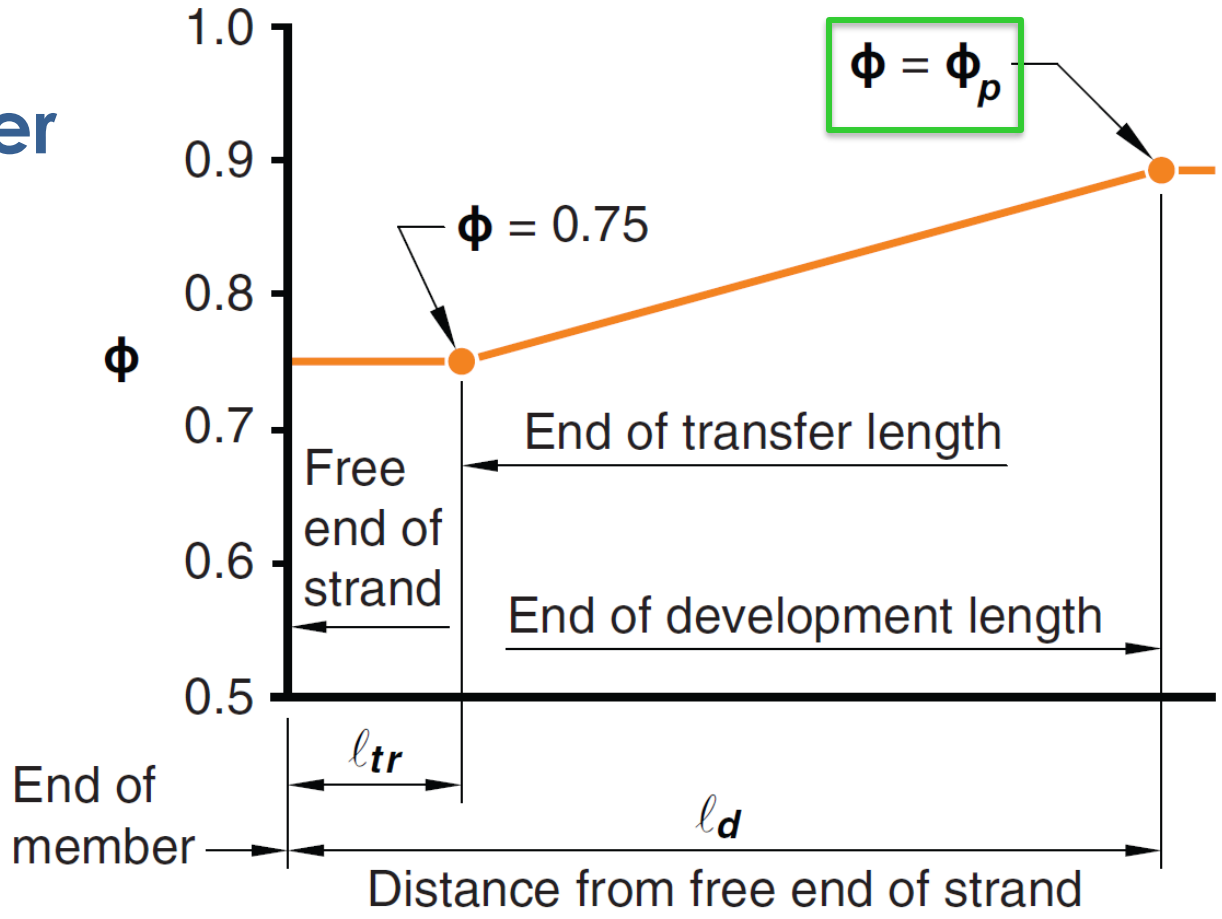


Fig. R16.5.1b

Strength Reduction Factor

Near end of precast member

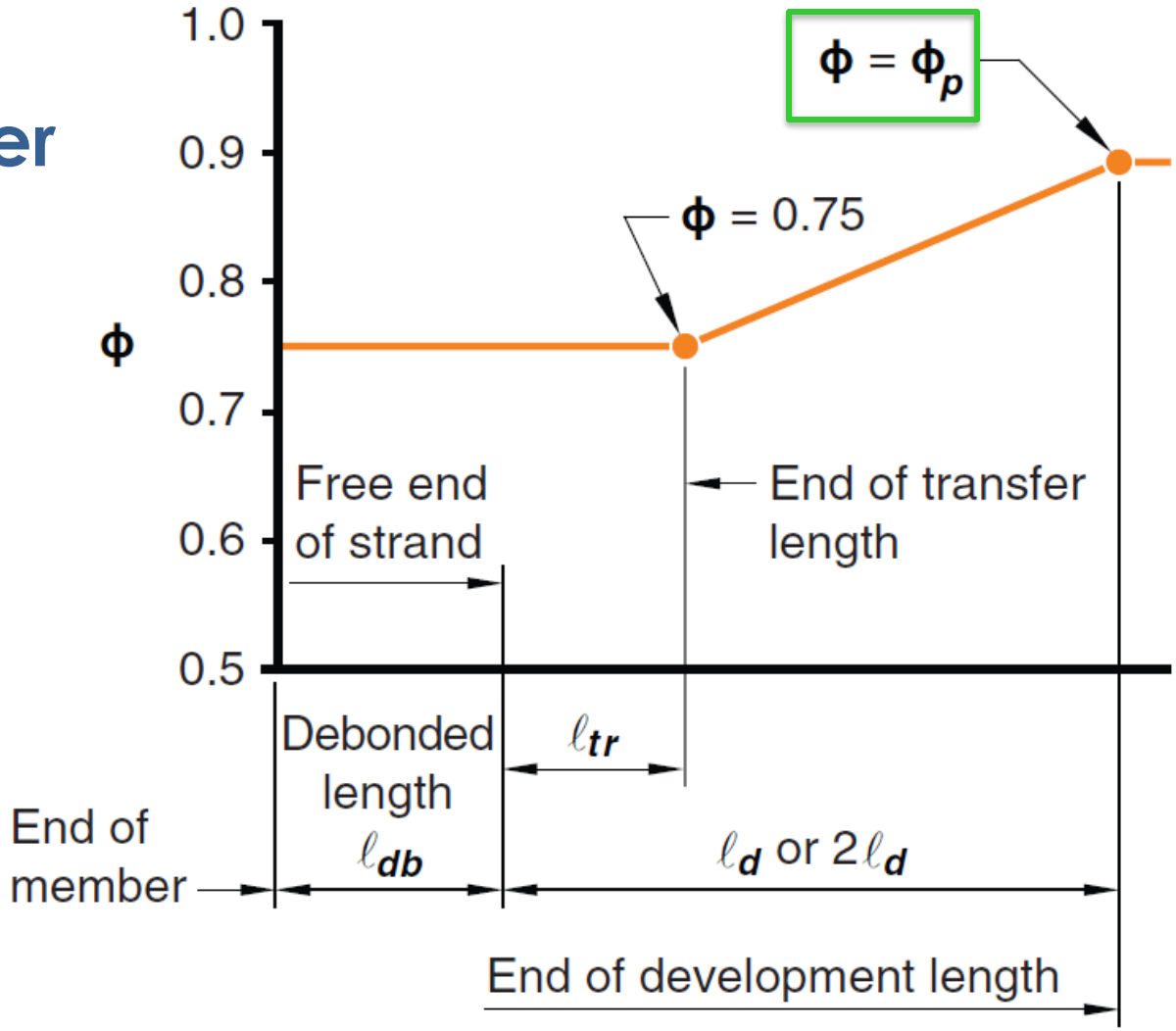
- Linear interpolation of ϕ
- ϕ_p depends on state of stress



Strength Reduction Factor

Near end of precast member

- Similar for debonded strand



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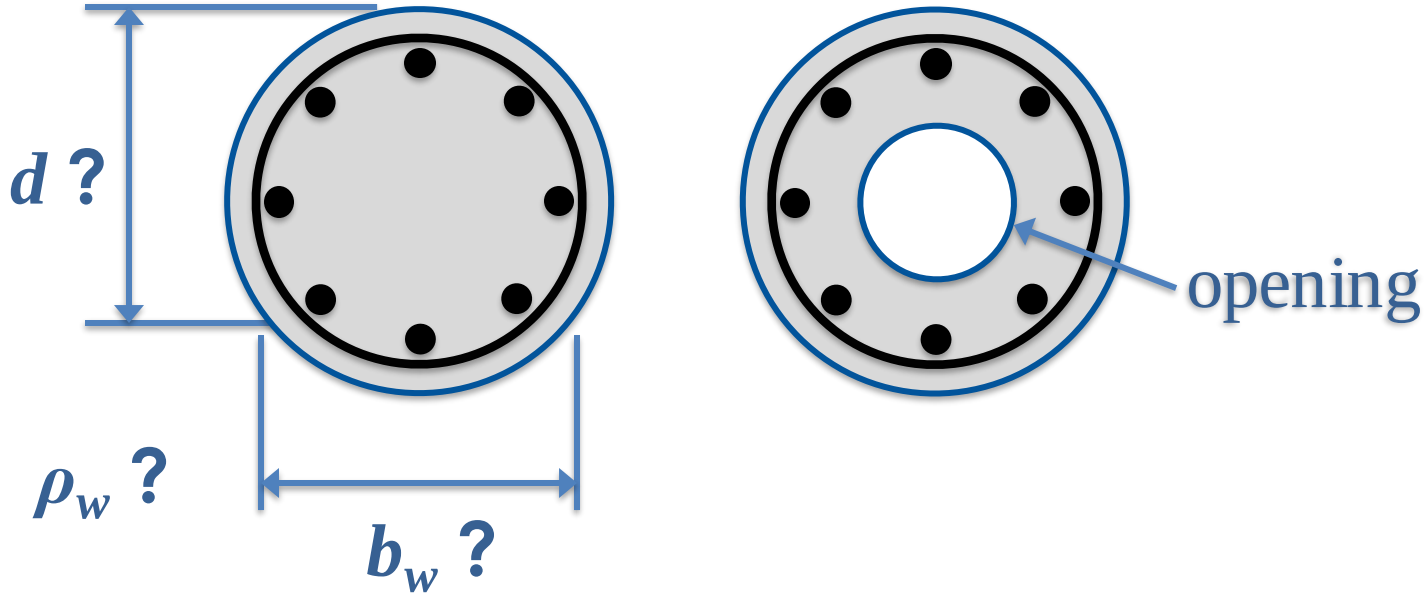
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Circular Sections

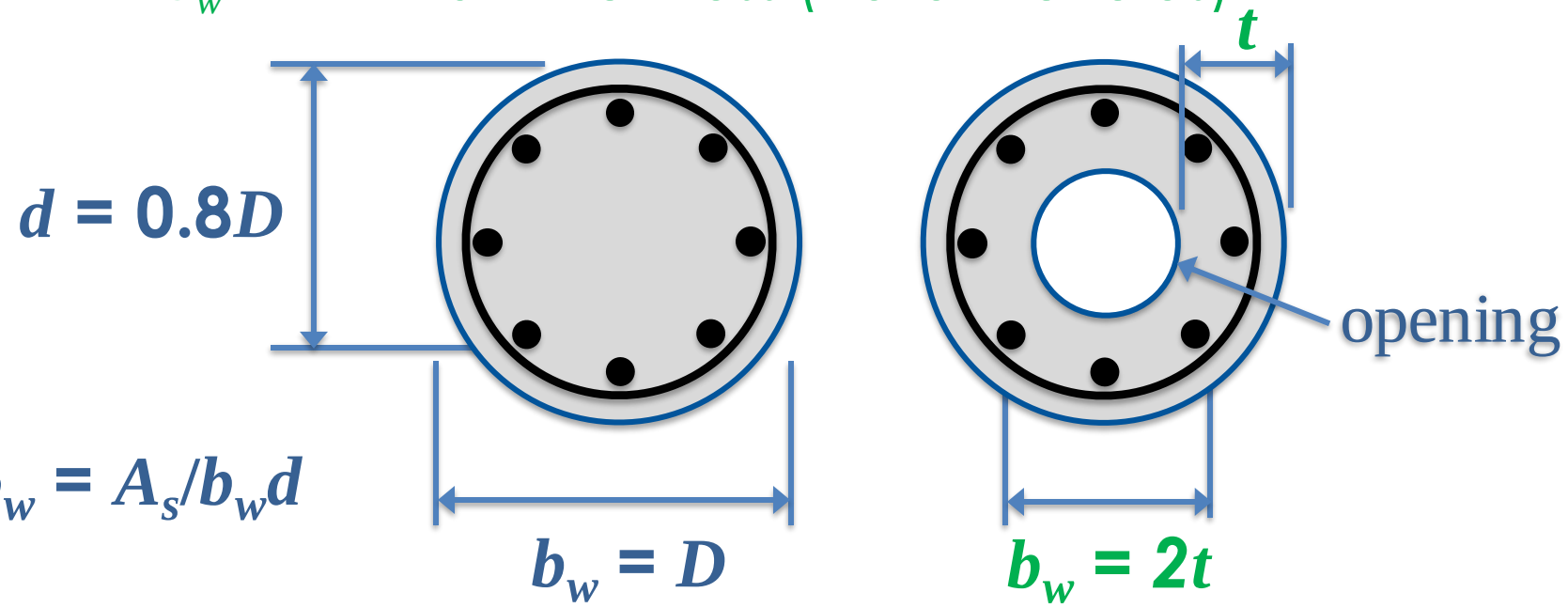
Variable definitions (22.5)

- **22.5 One-way shear**
  - Interpretation for hollow circular sections



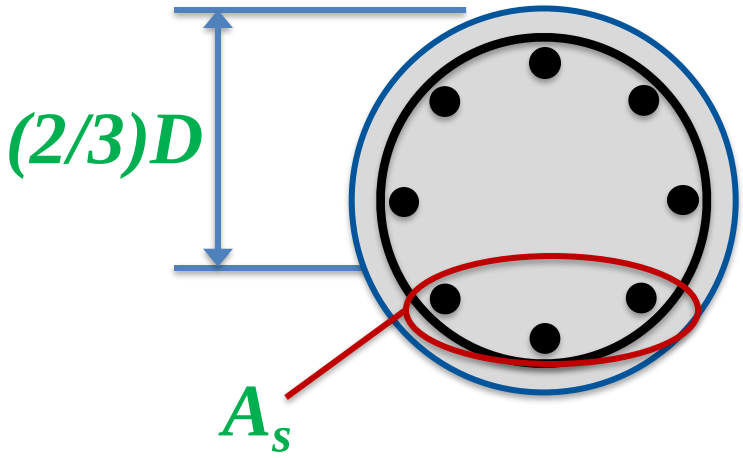
Variable definitions (22.5)

- **22.5.2.2 – calculation of V_c and V_s**
  - d = 0.8 x diameter
  - b_w = diameter (solid circles)
  - b_w = 2 x wall thickness (hollow circles)



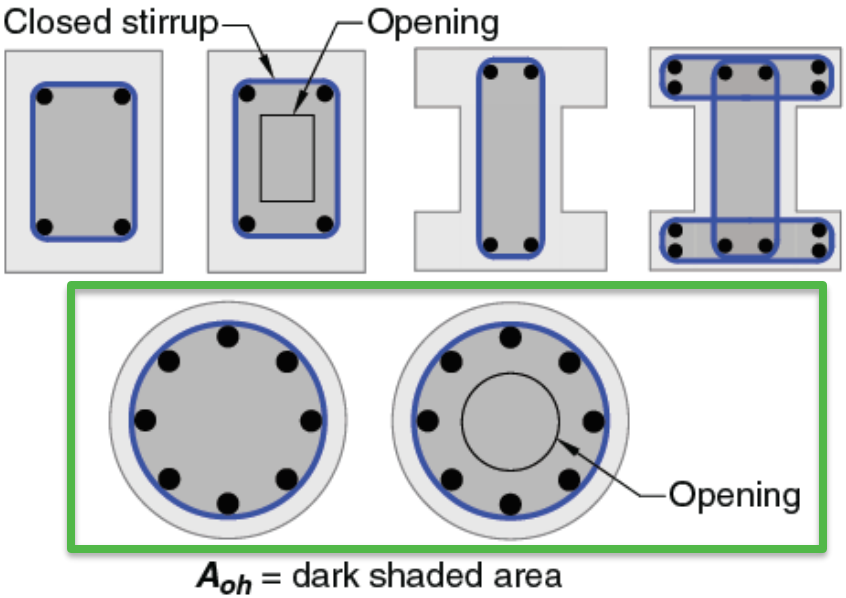
Variable definitions (22.5)

- What about A_s?



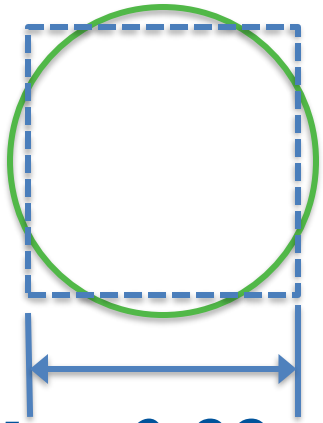
Torsion for circular sections (R22.7.6.1.1)

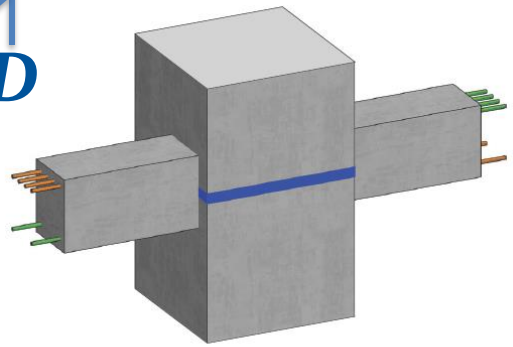
- Do ACI 318 torsion equations apply to circular cross sections?
- Code Eqns are based on thin-tube theory
- Examples added to figure

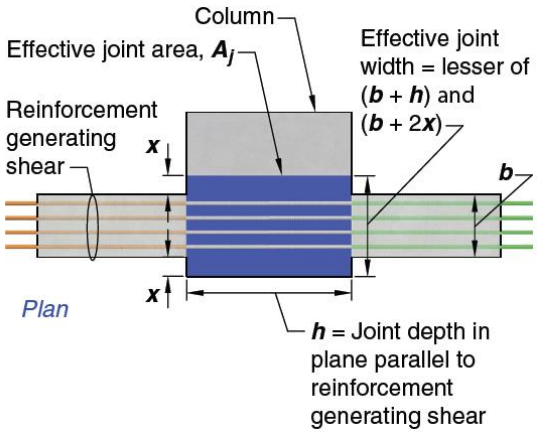


Circular Column Joints

- **Based on equivalent square column**
  - A_j for joint shear strength (15.4.2)
  - Width of transverse beams required for joint to be considered confined (15.2.8)
  - Column width ≥ 20 d_b for special moment frames (18.8.2.3)







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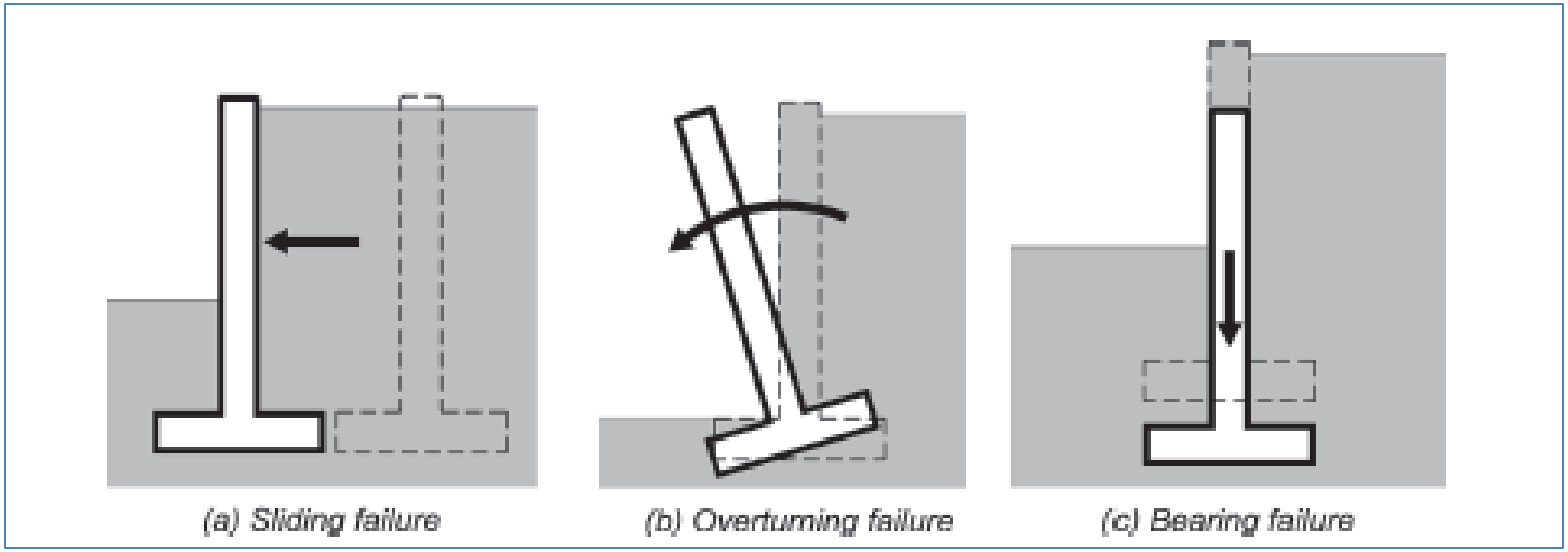


Walls

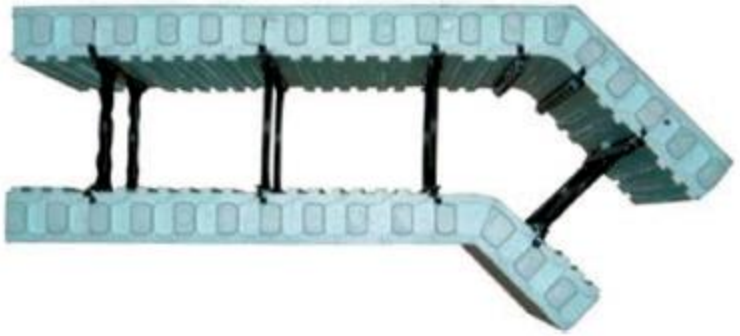
Scope of walls

- **Change in scope**

  11.1.4 - Design of cantilever retaining walls shall be in accordance with Chapter 13 (Foundations)



Scope of walls



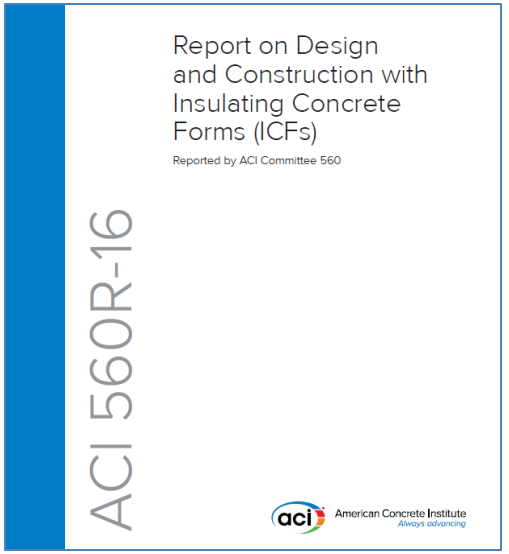
- **Added scope**

  11.1.6 - CIP walls with insulated forms shall be permitted by this code for use in one or two-story buildings

  - Design according to Chapter 11
  - Guidance – ACI 560R and PCA 100-2017
  - Unique construction issues



Photo courtesy Larry Novak

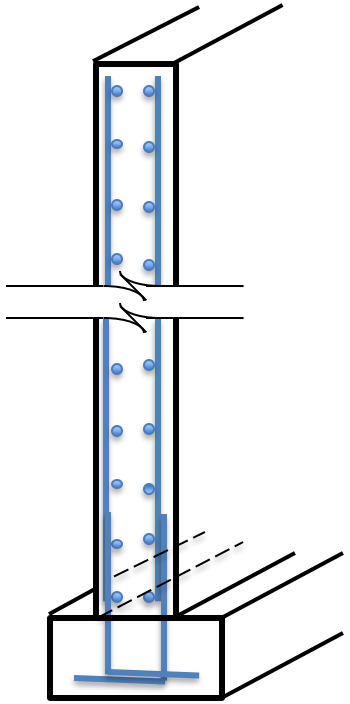


11.7.2.3 Bar placement

- **If wall thickness h > 10 in.**
  - Two layers of bars **one near each face**
  - Exception, single story basement walls

- **318-14**
  - ½ to 2/3 of reinf. placed near **exterior face**
  - Balance of reinf. placed near **interior face**

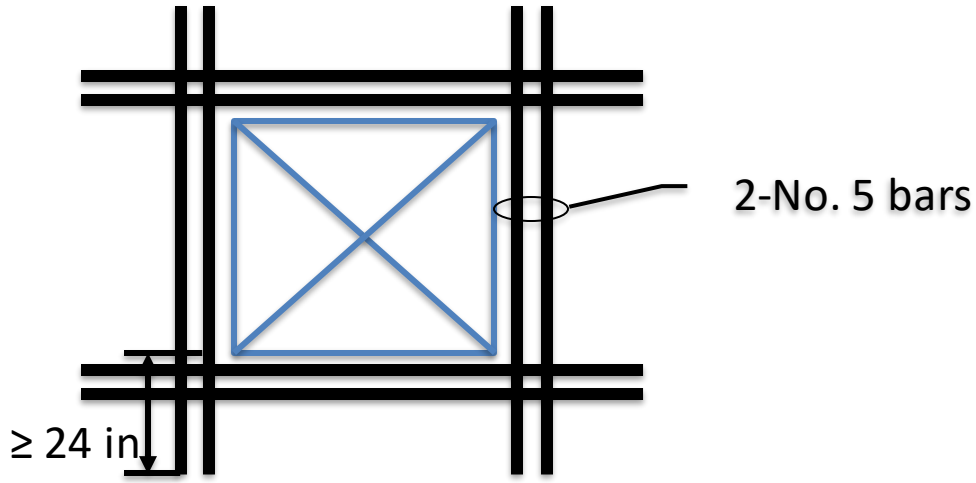
  - Confusion with exterior and interior
    - Face versus wall location
  - ½ to 2/3 was arbitrary



14.6 Plain concrete

At windows, door openings, and similarly sized openings

- At least two No. 5 bars (similar to walls 11.7.5.1)
- Extend 24 in. beyond or to develop f_y



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Foundations

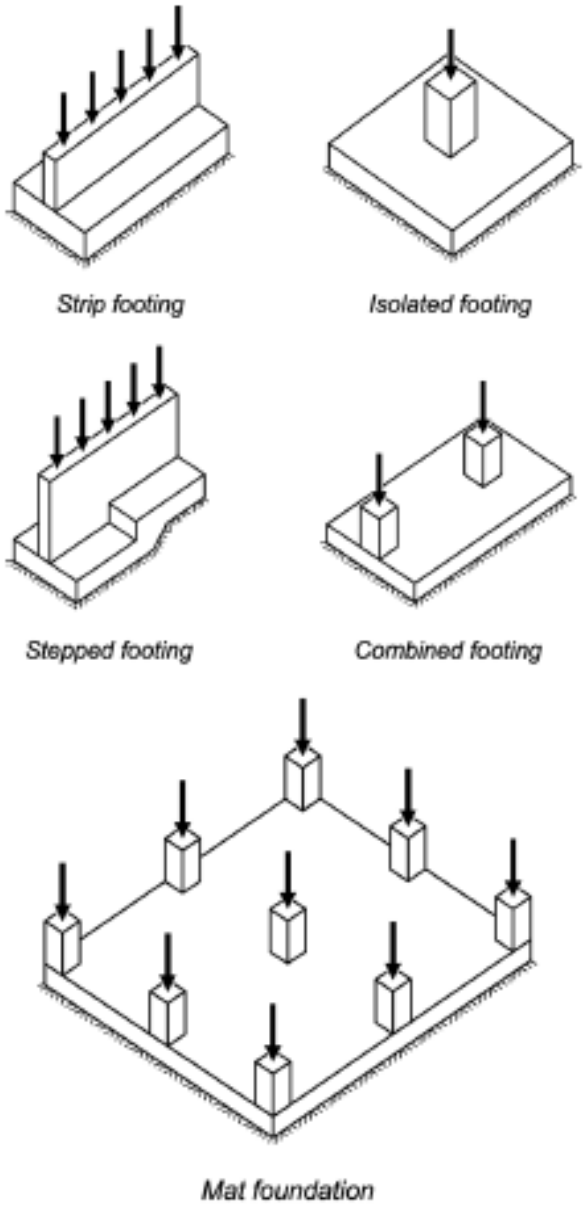


Ch. 13 – Foundations – significant changes

- **Added design provisions**
  - Cantilever retaining walls
  - Deep foundation design
- **Other**
  - Minimum concrete strengths for shallow and deep foundations
  - Cover

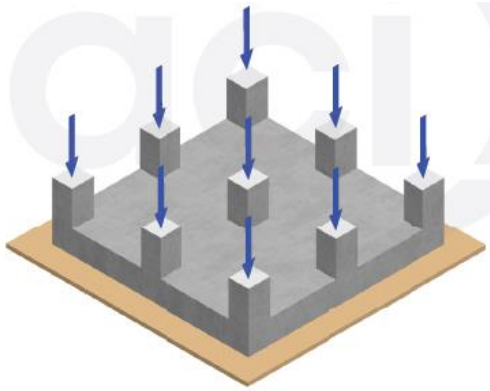
Foundations and 318

- **ACI 318-71 to ACI 318-11 (Ch. 15)**
  - Shallow footings, pile caps
- **ACI 318-14 (Ch. 13)**
  - Shallow footings, pile caps

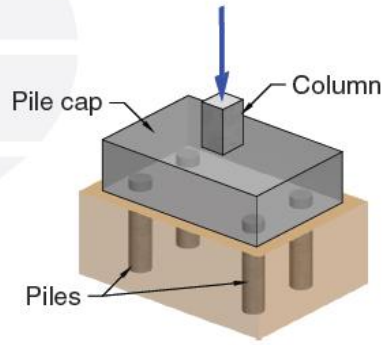


Foundations and 318

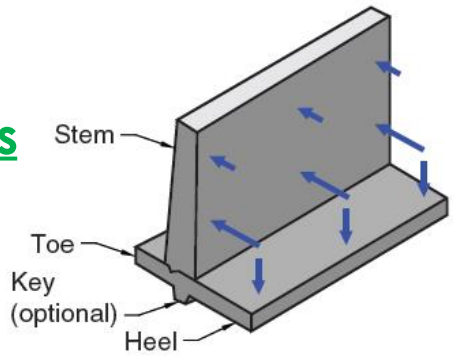
- **ACI 318-71 to ACI 318-11 (Ch. 15)**
  - Shallow footings, pile caps
- **ACI 318-14 (Ch. 13)**
  - Shallow footings, pile caps
- **ACI 318-19 (Ch. 13)**
  - **Shallow footings, pile caps, deep foundations, and walls of cantilevered retaining walls**

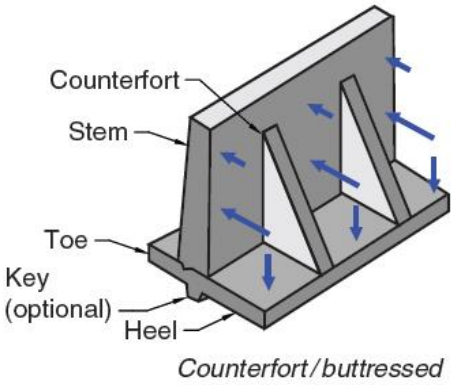


Mat foundation



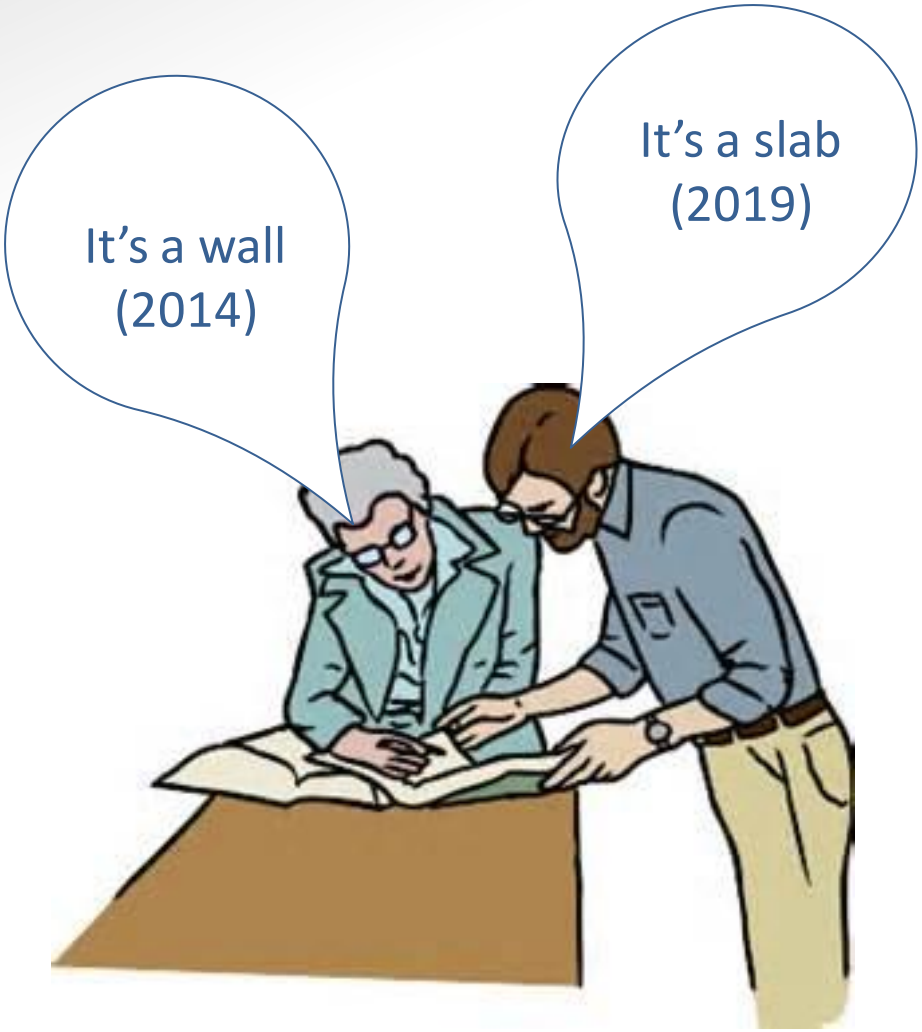
Deep foundation system with piles and pile cap

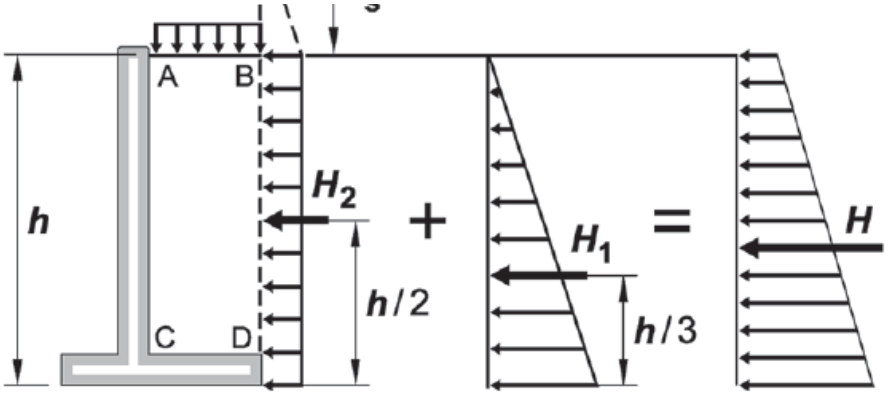




Counterfort/buttressed

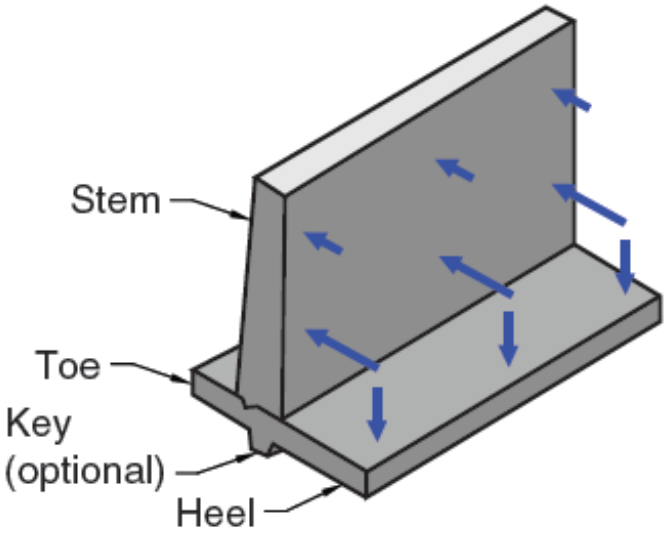
Cantilever retaining walls

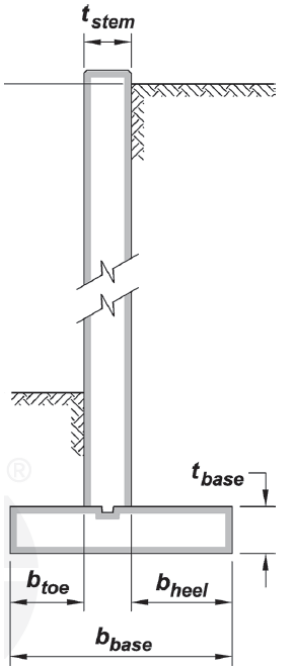




13.3.6.1—Cantilever stem walls

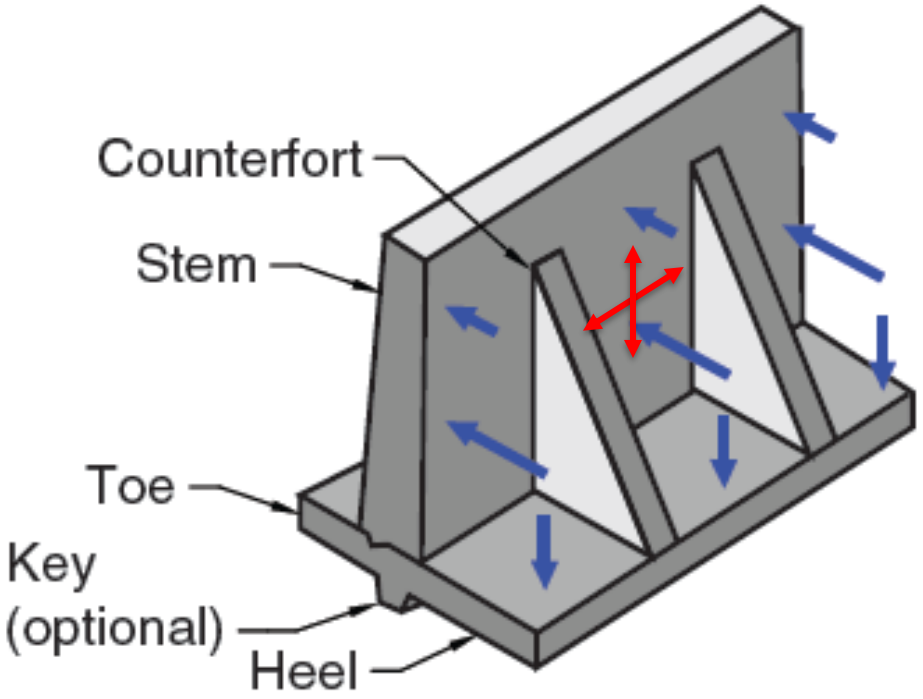
- Design as one-way slab (Ch. 7)





13.3.6.2—Cantilever stem wall with counterfort

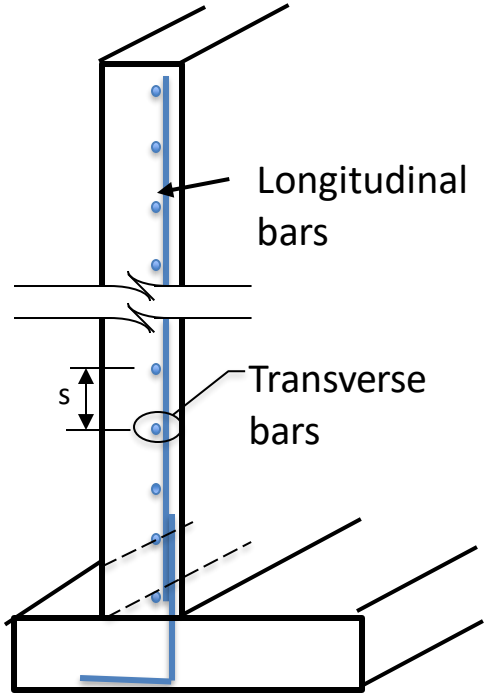
- Design as two-way slab (Ch. 8)



Counterfort/buttressed

Maximum bar spacing in stem wall

	Wall		Slab	
Stem wall reinforcement	Maximum bar spacing (2014)	Design as wall (2014)	Maximum bar spacing (2019)	Design as one-way slab (2019)
Long. (Wall) or Flexural (Slab)	3h, or 18 in.	11.7.2.1	Lesser of: $15\left(\frac{40,000}{f_s}\right) - 2.5c_c$ $12\left(\frac{40,000}{f_s}\right)$	7.7.2.2 (24.3)
Trans. (Wall) or S & T (Slab)	3h, or 18 in.	11.7.3.1	5h, or 18 in.	7.7.6.2.1



Minimum reinforcement in stem wall

ACI 318-14		ACI 318-19	
Minimum reinforcement, ρ	**Design as wall**	**Minimum reinforcement $A_{s,min}$**	**Design as one-way slab**
≤ No. 5 $\rho_\ell = 0.0012$ > No. 5 $\rho_\ell = 0.0015$	11.6.1	$A_{s,min} = 0.0018\ A_g$	7.6.1.1
≤ No. 5 $\rho_t = 0.0020$ > No. 5 $\rho_t = 0.0025$	11.6.2	$A_{S+T} = 0.0018\ A_g$	7.6.4.1 (24.4)

1.4.7— Scope changes – deep foundations

- **Scope:** This code does not govern design and installation of portions of concrete pile, drilled piers, and caissons **embedded in ground**, except as provided in (a) **through (c)**
  - (a) For portions in air or water, or in soil incapable of providing adequate lateral restraint to prevent buckling throughout their length
  - **(b) For precast concrete piles supporting structures assigned to SDC A and B**
  - (c) **For deep foundation elements** supporting structures assigned to **SDC C**, D, E, and F **(SDC C is added to scope)**

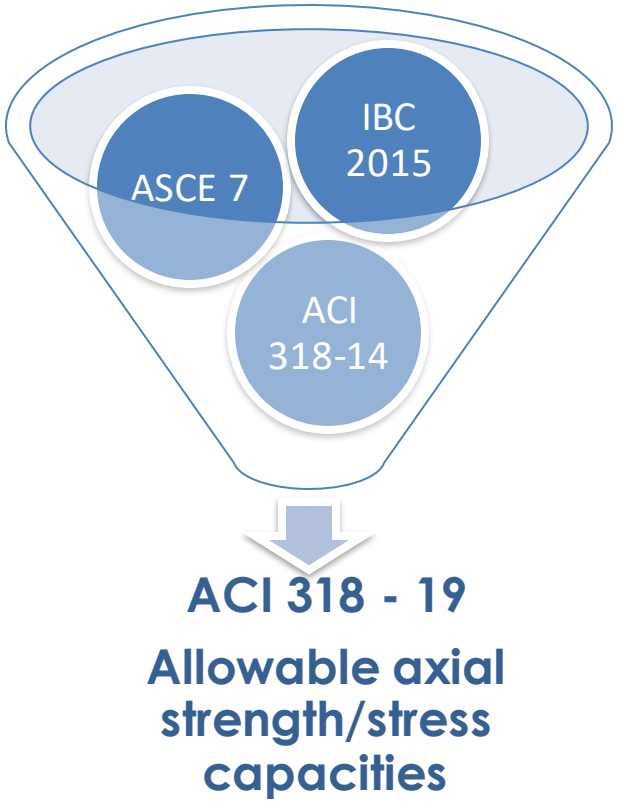


Deep Foundations (13.4)

- 13.4.1 General
- 13.4.2 Allowable axial strength
- 13.4.3 Strength design
- 13.4.4 Cast-in-place deep foundations
- 13.4.5 Precast concrete piles
- 13.4.6 Pile caps

Deep foundation – combine IBC & ASCE 7

- **ACI 318 – 19 –**
  - combined **IBC 2015**, **ASCE 7-10**, and **ACI 318-14** with regards to design of deep foundations for earthquake resistant structures (SDC C, D, E, and F)



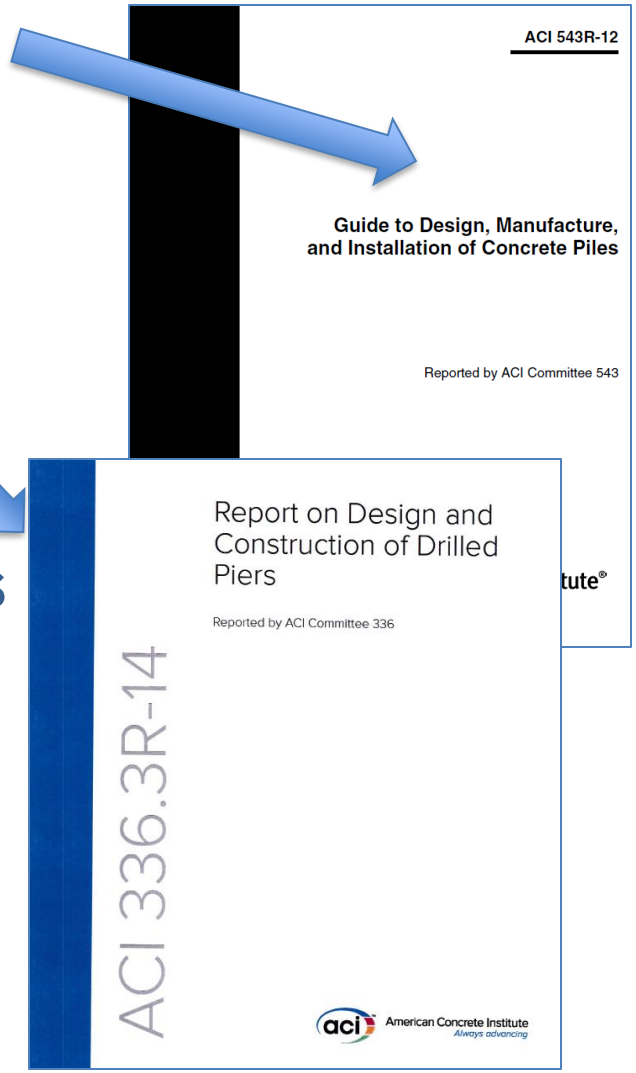
Pre- ACI 318-19 – design of deep foundations

- **ACI 543 - Piles (diam. < 30 in.)**
- **ACI 336.3 - Design of drilled piers (diam. ≥ 30 in.)**

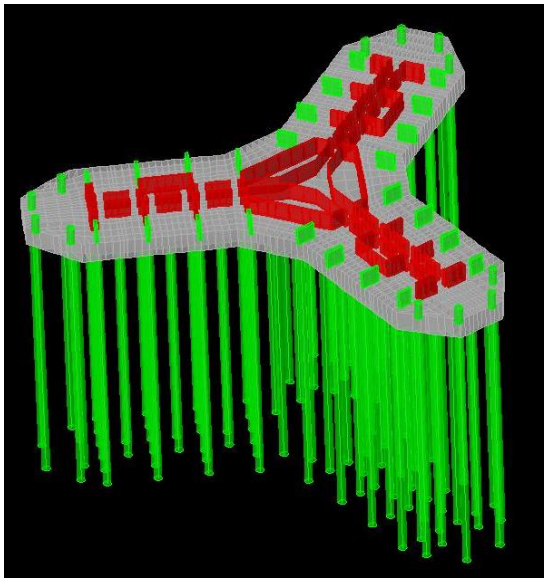
Not code language documents

Also used deep footing provisions from:

IBC and ASCE/SEI 7



Design of deep foundation members- compressive axial force (13.4.1)



- **Design axial strength of members in accordance to two methods:**
  - Allowable Axial Strength Design (13.4.2)
  - Strength Design (13.4.3)





Photos courtesy Larry Novak

Allowable axial strength method (13.4.2)

13.4.2.1 It shall be permitted to design a deep foundation member using load combinations for **allowable stress design in ASCE / SEI 7**, Section 2.4, and the **allowable strength specified in Table 13.4.2.1** if (a) and (b) are satisfied

(a) Deep foundation is **laterally supported** for its entire height

(b) Applied forces causing bending moments less than moment due to an accidental eccentricity of 5 percent of the pile diameter or width.

13.4.2 deep foundation design

Table 13.4.2.1—Maximum allowable compressive strength for deep foundation members

Deep foundation member type	Maximum allowable compressive strength [1]	
Uncased cast-in-place concrete drilled or augered pile	$P_a = 0.3f_c'A_g + 0.4f_yA_s$	(a)
Cast-in-place concrete pile in rock or within a pipe, tube, or other permanent metal casing that does not satisfy 13.4.2.3	$P_a = 0.33f_c'A_g + 0.4f_yA_s$ [2]	(b)
Metal cased concrete pile confined in accordance with 13.4.2.3	$P_a = 0.4f_c'A_g$	(c)
Precast nonprestressed concrete pile	$P_a = 0.33f_c'A_g + 0.4f_yA_s$	(d)
Precast prestressed concrete pile	$P_a = (0.33f_c' - 0.27f_{pc})A_g$	(e)

[1] A_g applies to the gross cross-sectional area. If a temporary or permanent casing is used, the inside face of the casing shall be considered the concrete surface.

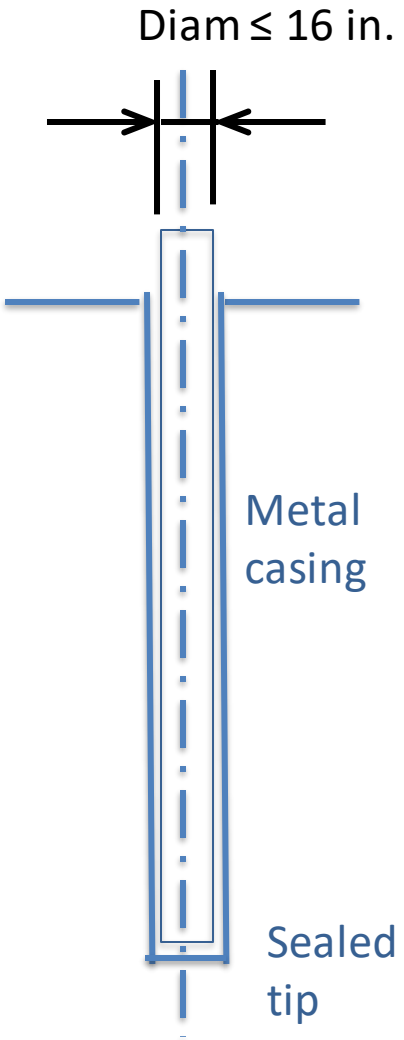
[2] A_s does not include the steel casing, pipe, or tube.

Confinement of metal casing (13.4.2.3):

- not used to resist axial load
- sealed tip and mandrel-driven
- seamless or welded seamless

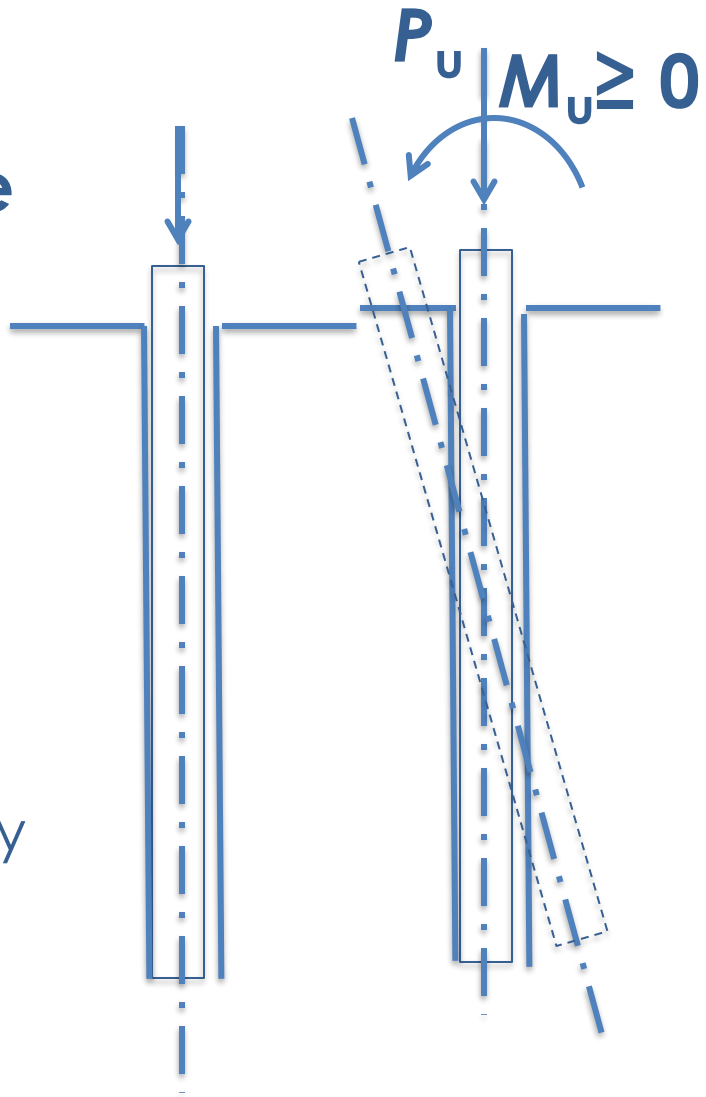
Physical properties

- wall thickness ≥ 14 ga. (0.068 in.)
- $f_y \geq 30{,}000$ psi
- $f_y \geq 6\, f'_c$, and
- nominal diameter ≤ 16 in.



Deep foundations – strength design (13.4.3)

- **Method may be used any time**
- **Method must be used when pile does not meet criteria for allowable axial strength design**
  - Soils do not provide lateral support
  - Moment is not negligible
- **Use Section 10.5 (columns)**
  - $\phi P_n \geq P_u$
  - $\phi M_n \geq M_u$
  - Combined P_n and M_n calculated by 22.4



Strength design (13.4.3) – axial force, no moment

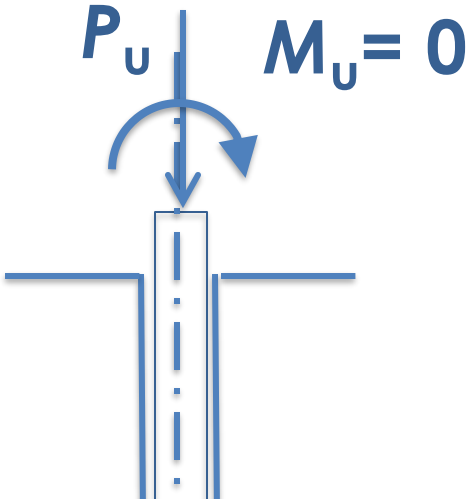
Nominal axial compressive strength; P_n

$$\phi P_{n,max} \geq P_u$$

Maximum axial strength

- For deep foundations members with ties conforming to Ch. 13 (new in Table 22.4.2.1)

$$P_{n,max} = 0.80\, P_o$$



Where:

P_o = nominal axial strength at zero eccentricity

$$P_o = 0.85 f'_c (A_g - A_{st}) + f_y A_{st}$$

Table 22.4.2.1—Maximum axial strength

Member	Transverse reinforcement	$P_{n,max}$	
Nonprestressed	Ties conforming to 22.4.2.4	$0.80P_o$	(a)
	Spirals conforming to 22.4.2.5	$0.85P_o$	(b)
Prestressed	Ties	$0.80P_o$	(c)
	Spirals	$0.85P_o$	(d)
Deep foundation member	Ties conforming to Ch. 13	$0.80P_o$	(e)

Strength design (13.4.3) – axial force, no moment

- **Reduction factor – Table 13.4.3.2**

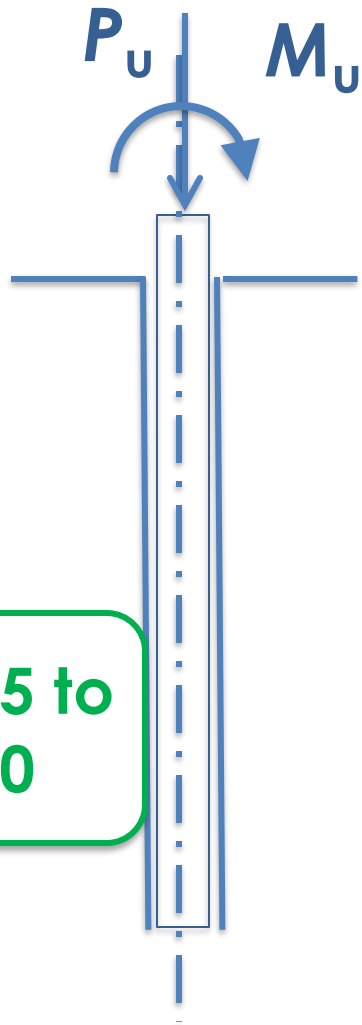


Table 13.4.3.2—Compressive strength reduction factors ϕ for deep foundation members

Deep foundation member type	Compressive strength reduction factors ϕ	
Uncased cast-in-place concrete drilled or augered pile[1]	0.55	(a)
Cast-in-place concrete pile in rock or within a pipe, tube,[2] or other permanent casing that does not satisfy 13.4.2.3	0.60	(b)
Cast-in-place concrete-filled steel pipe pile[3]	0.70	
Metal cased concrete pile confined in accordance with 13.4.2.3	0.65	
Precast-nonprestressed concrete pile	0.65	(e)
Precast-prestressed concrete pile	0.65	(f)

0.55 to 0.70

Deep foundations

13.4.4.1 CIP deep foundations that are subject to (a) **uplift** or (b) $M_u > 0.4M_{cr}$ shall be reinforced, unless enclosed by a steel pipe or tube

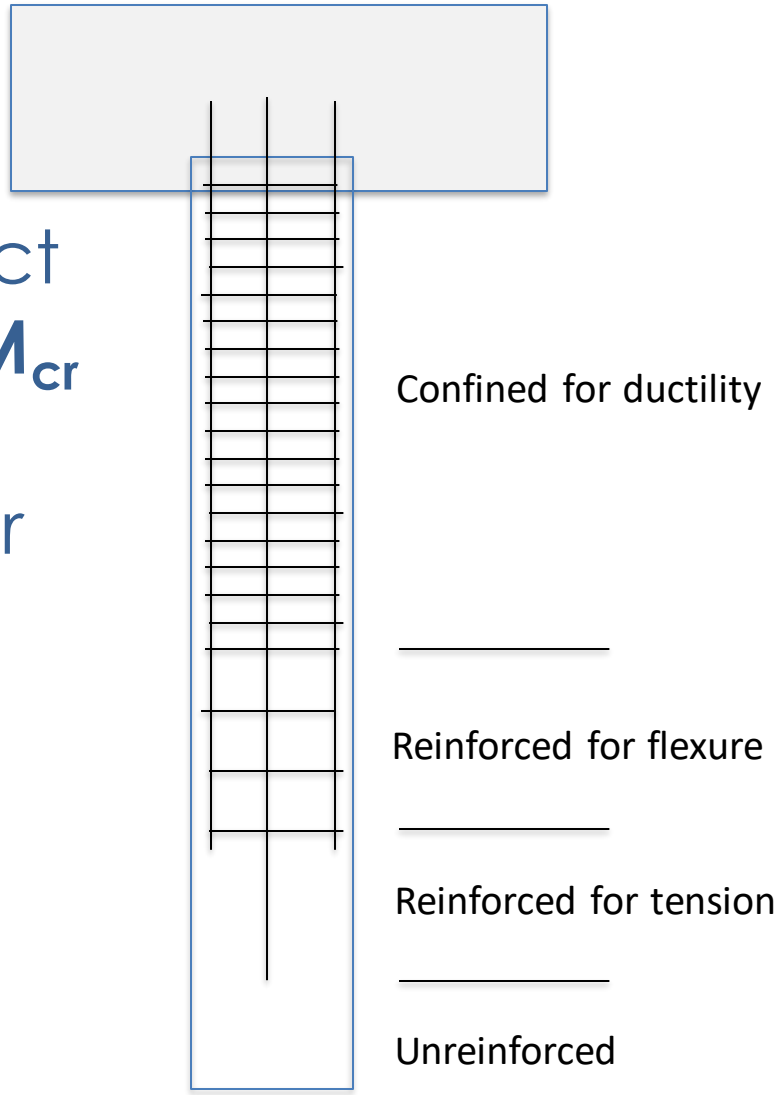
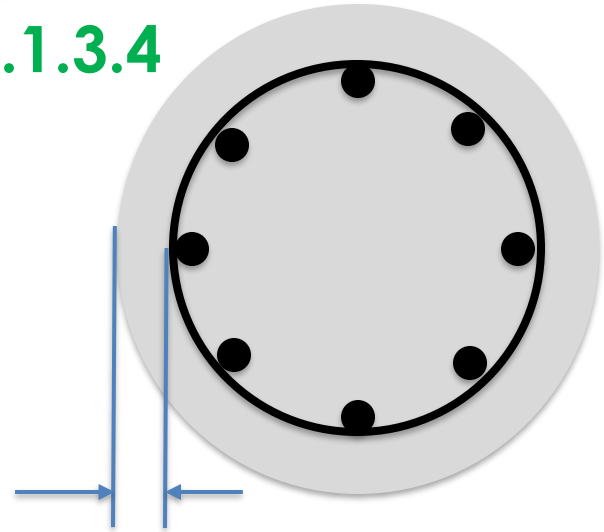


Table 19.2.1.1 – Additional minimum strength, f'_c

Shallow foundations	Min. f'_c (psi)
Foundations in SDC A, B, or C	2500
Foundation for Residential and Utility …. 2 stories or less ….stud bearing construction …… SDC D, E, or F	2500
Foundation for Residential and Utility …. More than 2 stories….stud bearing construction …… SDC D, E, or F	3000
Deep foundations	
Drilled shafts or piers	4000
Precast nonprestressed driven piles	4000
Precast prestressed driven piers	5000

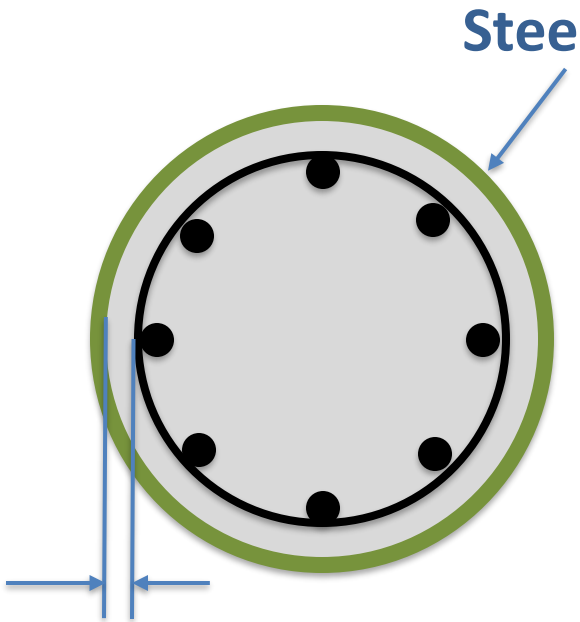
Concrete cover – deep foundations

Table 20.5.1.3.4



3 in.

Cast-in-place against ground

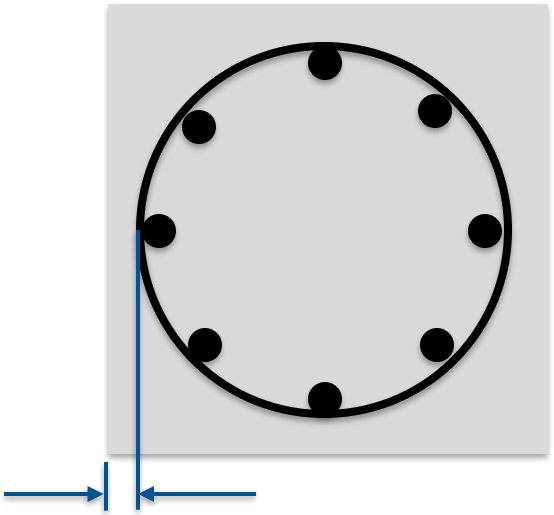


1.5 in.

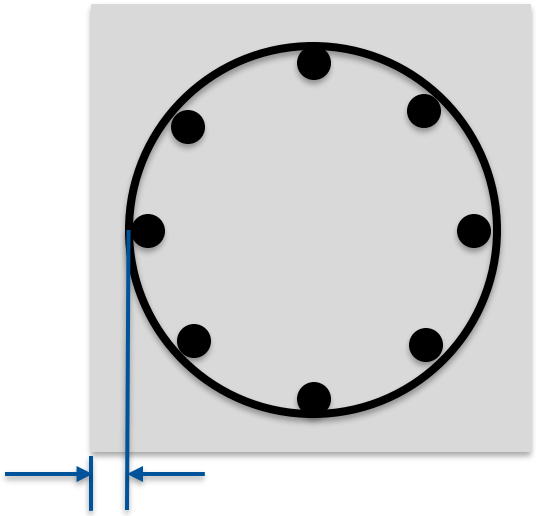
Cast-in-place enclosed by steel pipe, permanent casing, or stable rock socket

Concrete cover – deep foundations

Table 20.5.1.3.4



1.5 in. precast nonprestressed
and precast prestressed



2.5 in. precast nonprestressed
2 in. precast prestressed

In contact with ground

Exposed to seawater

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Anchorage to Concrete

Chapter 17 – Anchoring to Concrete

- **Reorganized**
- **New content/design information**
  - Screw anchors added
  - Shear lugs added



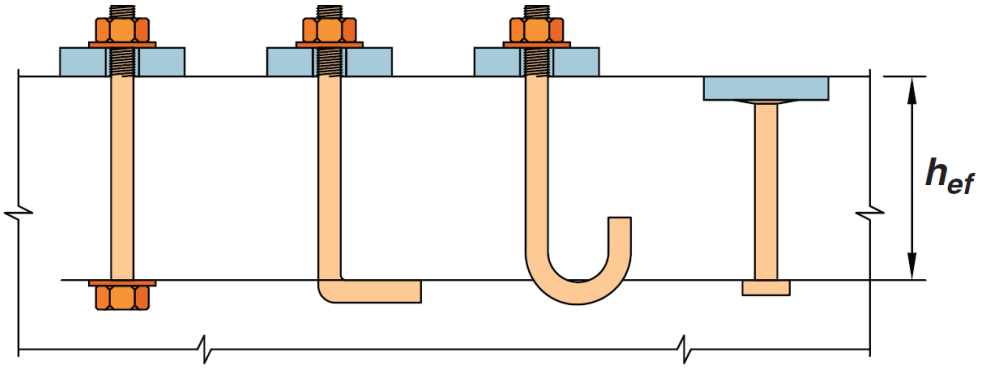
Sections

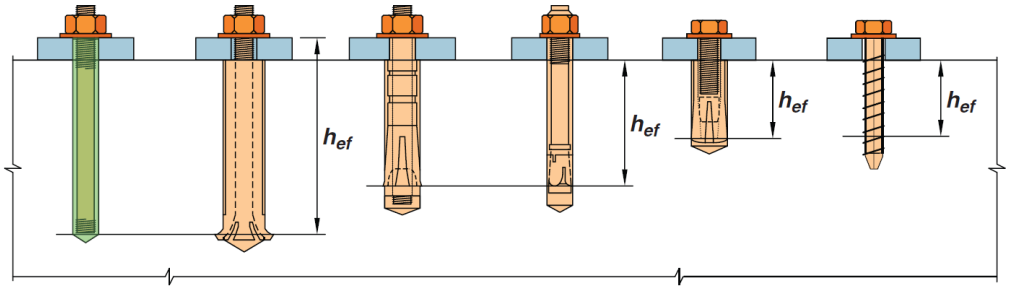
- 17.1 Scope
- 17.2 General
- 17.3 Design limits
- 17.4 Required strength
- 17.5 Design strength
- 17.6 Tensile strength
- 17.7 Shear strength
- 17.8 Tension and shear interaction
- 17.9 Edge distances, spacings, and thicknesses to preclude splitting failure
- 17.10 Earthquake-resistant design requirements
- 17.11 Attachments with shear lugs

Ch. 17 – Anchoring to Concrete

Scope

- Headed studs and headed bolts
- Hooked bolts
- Post-installed undercut anchors
- Post-installed expansion anchors
- Post-installed adhesive anchors



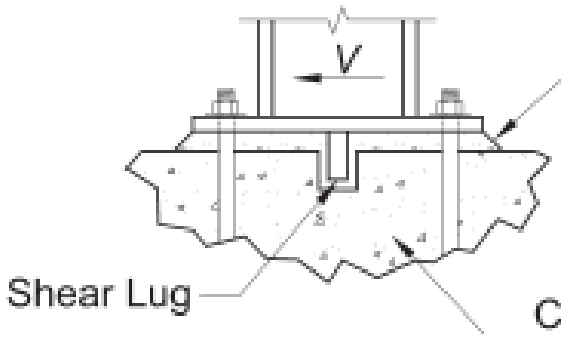


New Content/Design Information

- **Post-installed screw anchors**
  - pre-qualification per ACI 355.2
- **Attachments with shear lugs**

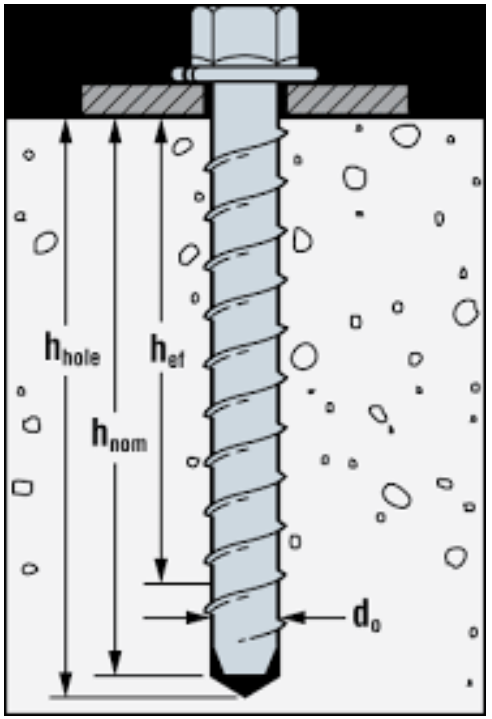






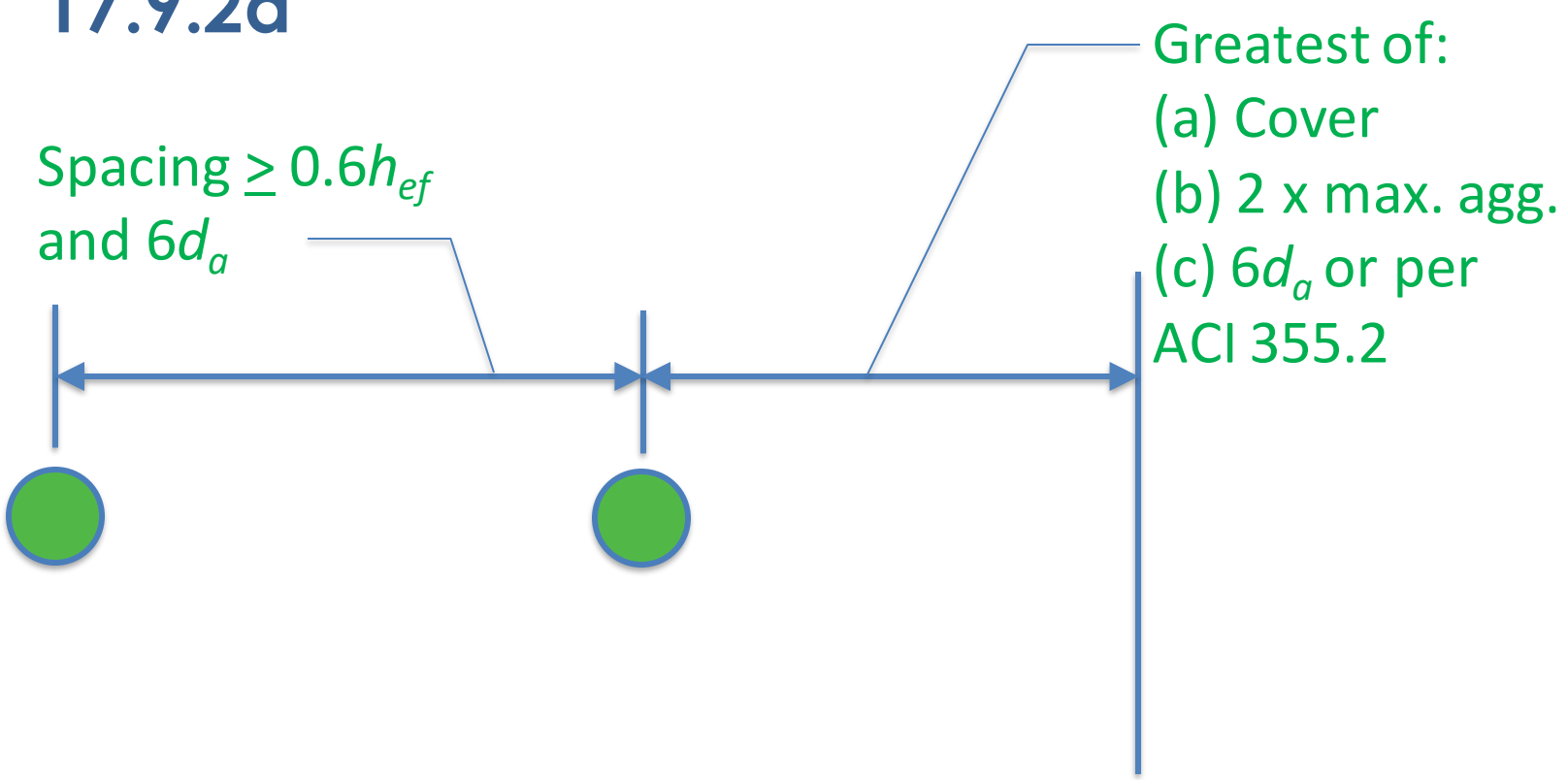
Screw Anchors (17.3.4)

- **For screw anchors satisfying:**
  - $h_{ef} \geq 1.5$ in. and
  - $5d_a \leq h_{ef} \leq 10d_a$
- **Manufacturer provides h_{ef}, A_{ef}, and pullout strength**
- **Concrete breakout evaluated similar to other anchors**
  - 17.6.2 in tension
  - 17.7.2 in shear



Minimum Spacing (17.9.2a)

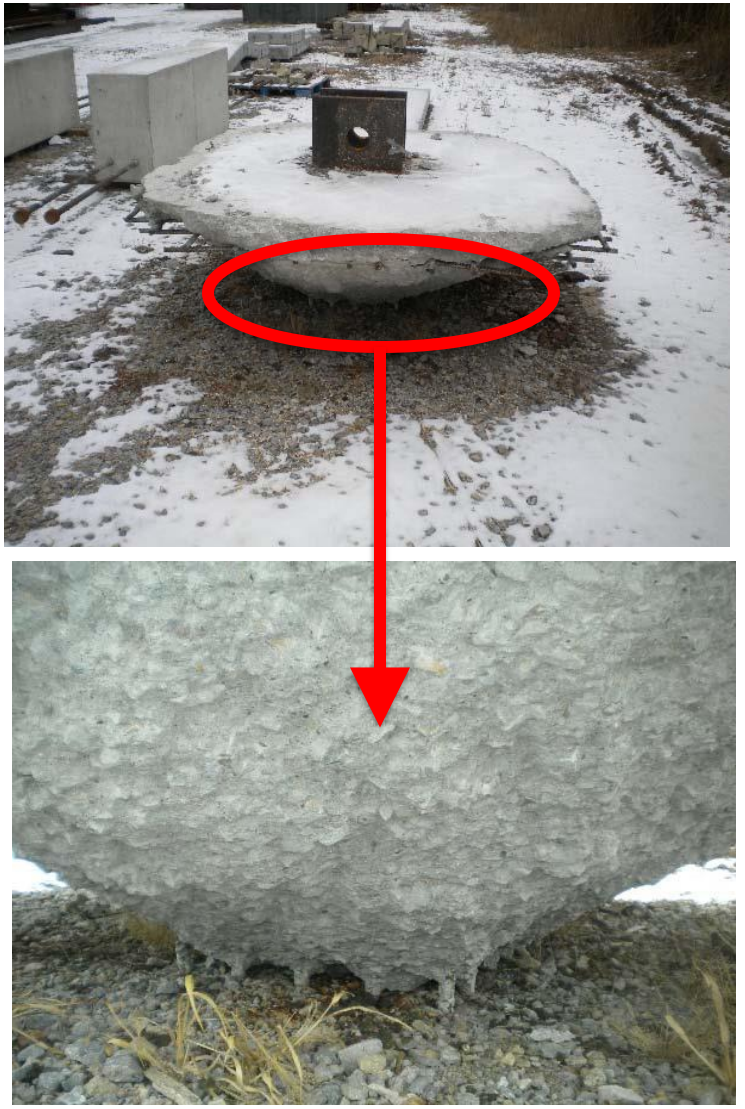
- Screw anchor spacing limited per Table 17.9.2a



17.1.6 – Reinforcement used as anchorage

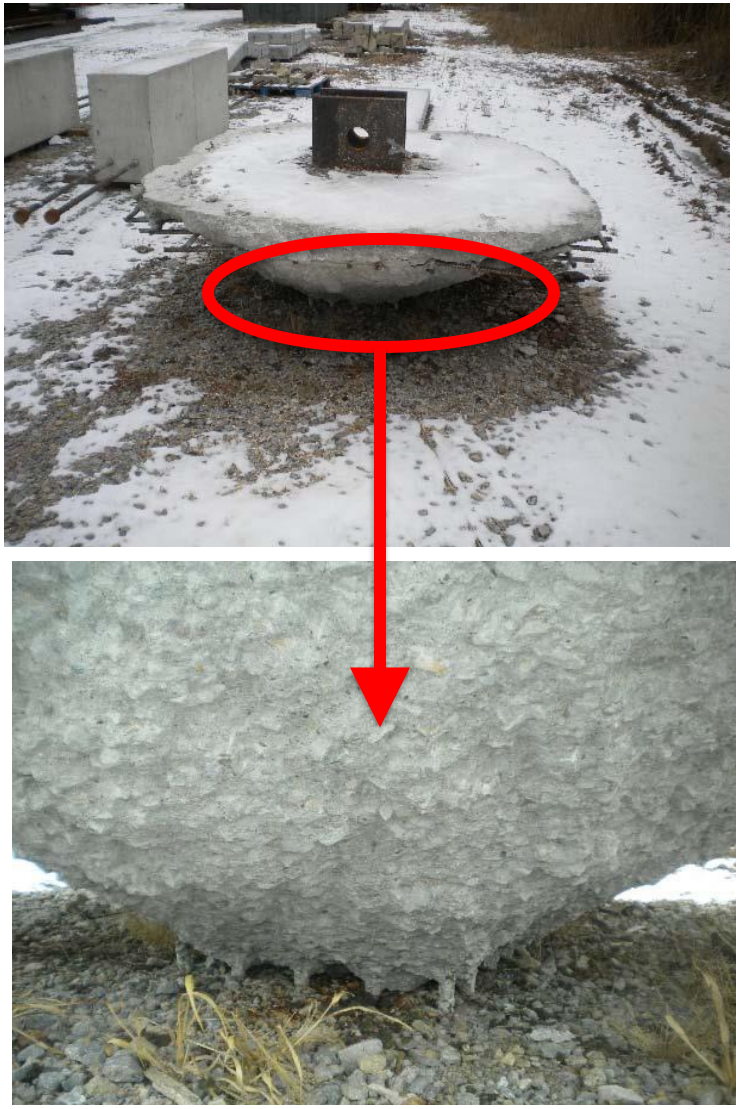
Check anchorage for bars developed per Ch. 25

- Check concrete breakout in tension (and maybe shear)
- Greater development length should be considered



17.1.6 – Reinforcement used as anchorage

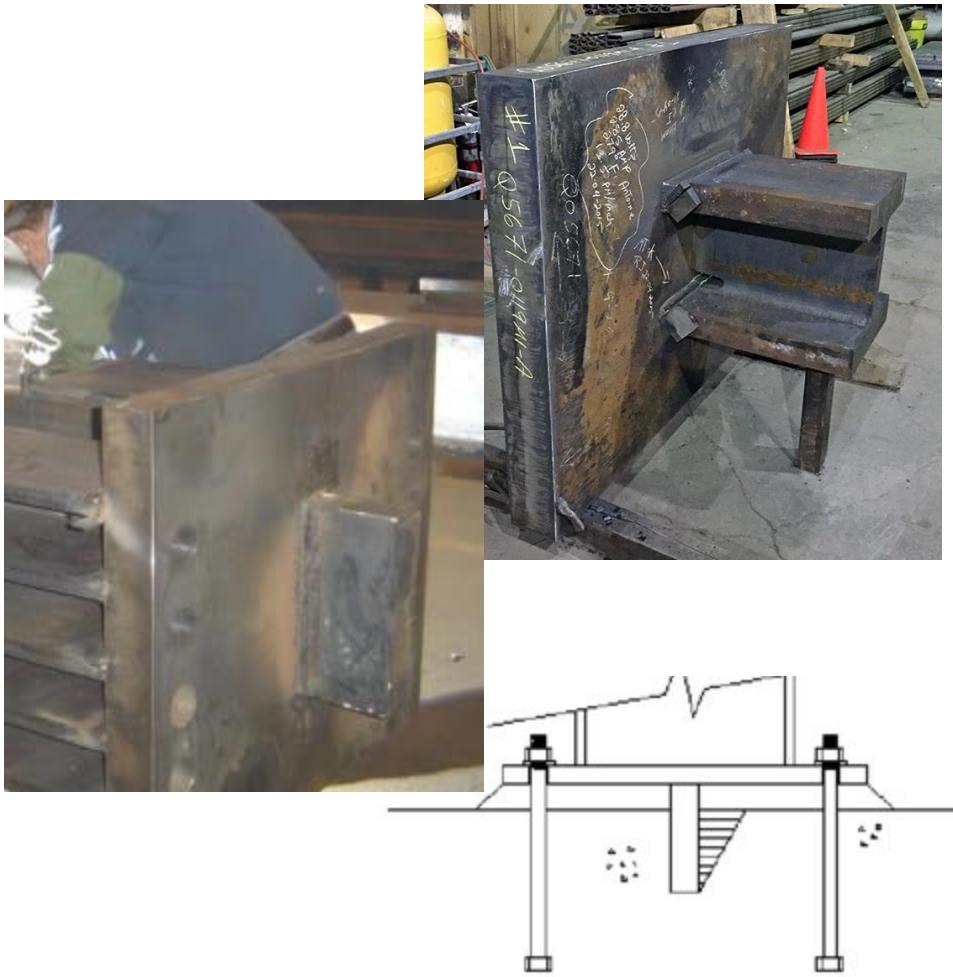
- Straight bars behave like adhesive anchors
- Hooked and headed bars behave like headed anchors
- Anchor reinforcement may be an alternative



Shear Lugs (17.11.1)

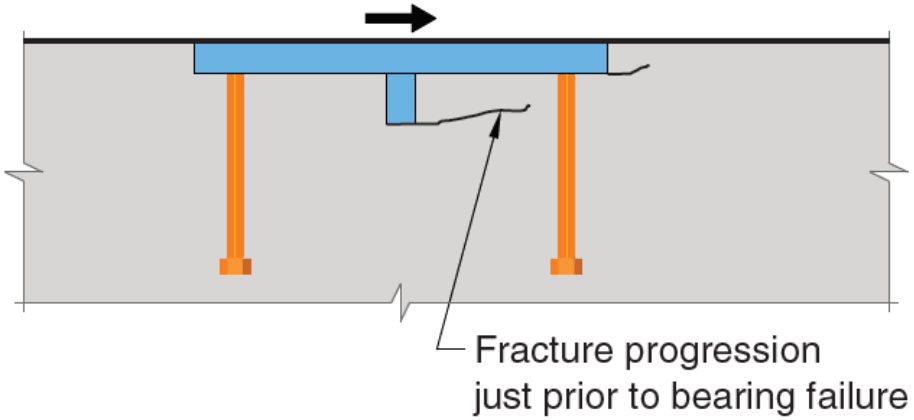
Shear lugs are fabricated from:

- Rectangular plates or
- Steel shapes composed of plate-like elements, welded to an attachment base plate

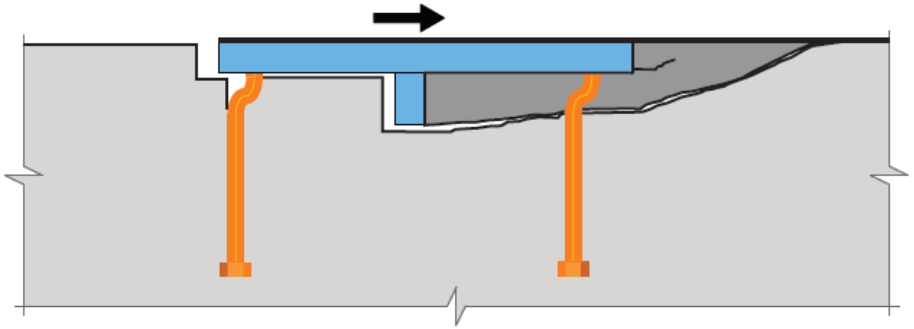


Shear Lugs (17.11.1)

- Minimum four anchors
- Anchors do not need to resist shear forces if not welded
- Anchors welded to steel plate carry portion of total shear load



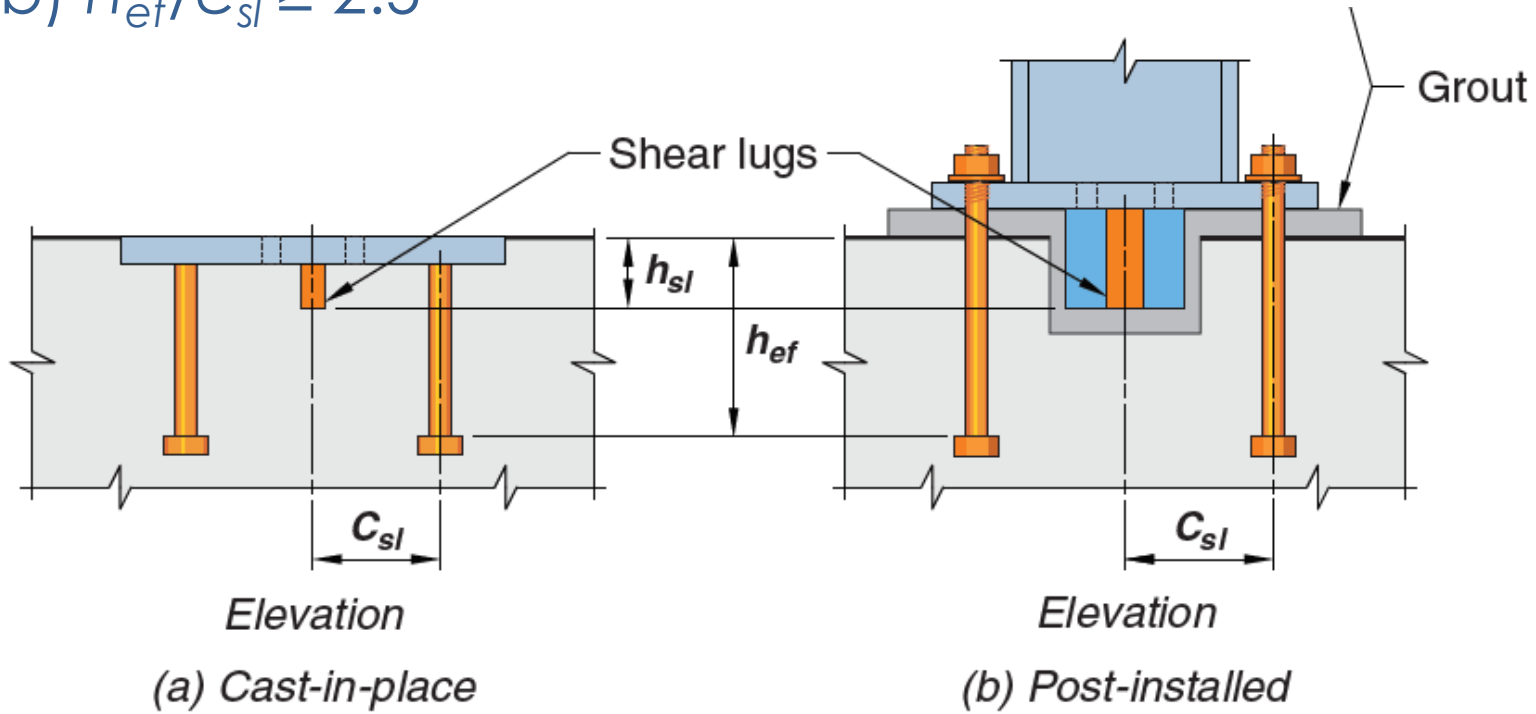
(a) Just prior to bearing failure



(b) Just prior to anchor steel failure

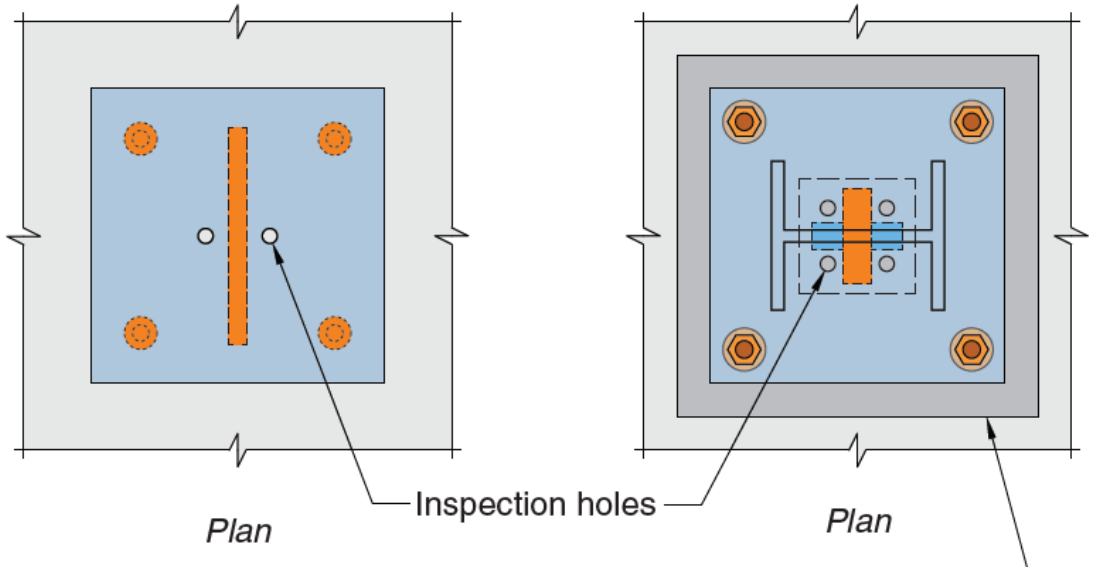
Shear Lug Detailing (17.11.1.1.8)

- **Anchors in tension, satisfy both (a) and (b):**
  (a) $h_{ef}/h_{sl} \geq 2.5$
  (b) $h_{ef}/c_{sl} \geq 2.5$

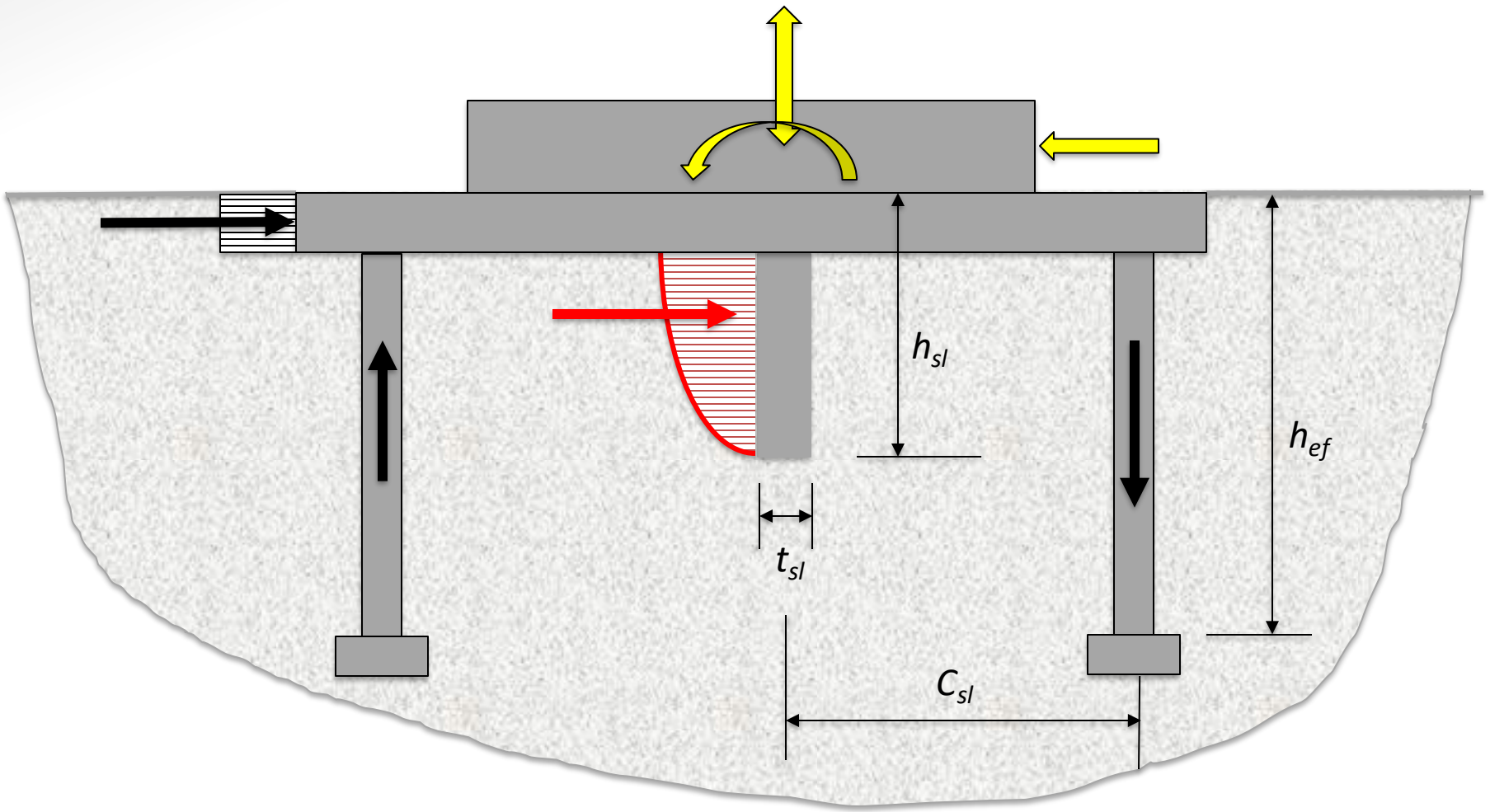


Shear Lug Detailing (17.11.1.2)

- Steel plate to have 1 in. dia. (min.) hole
- Single plate – one on each side
- Cross / cruciform plate - one each quadrant
- More vent holes are not detrimental

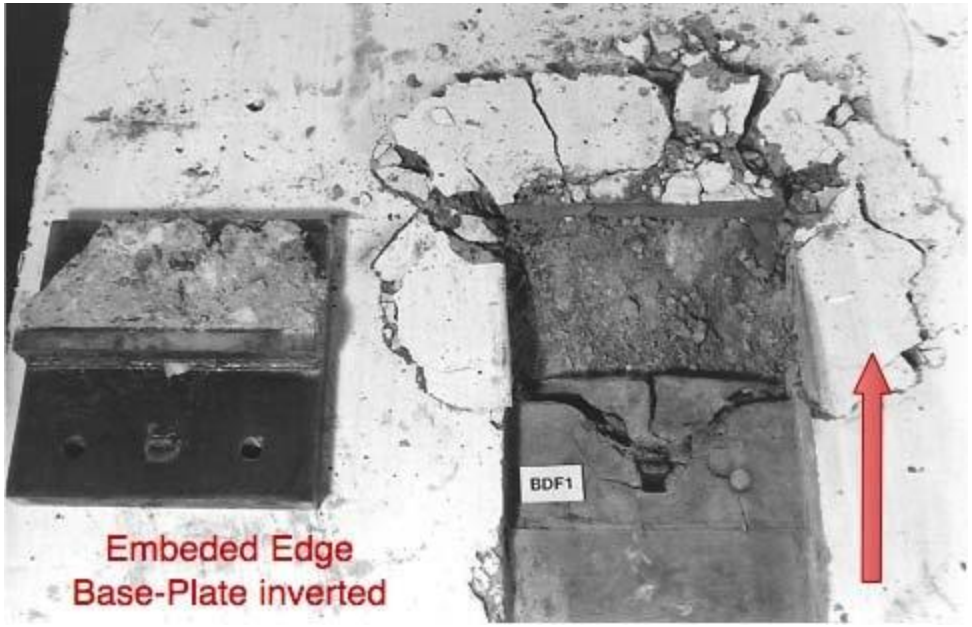


Shear Lug Overturning (17.11.1.1.9)

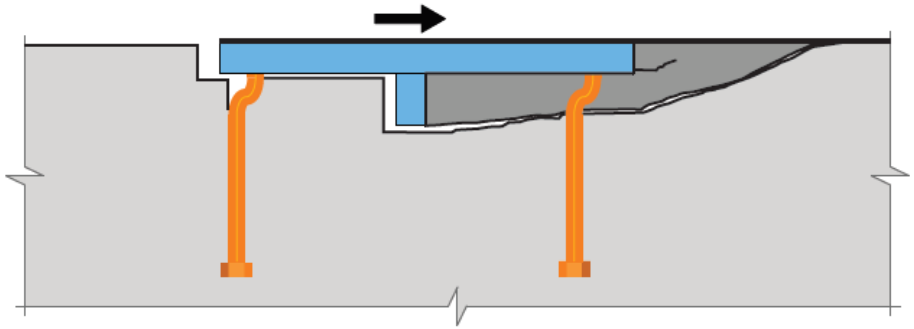


Bearing (17.11.2)

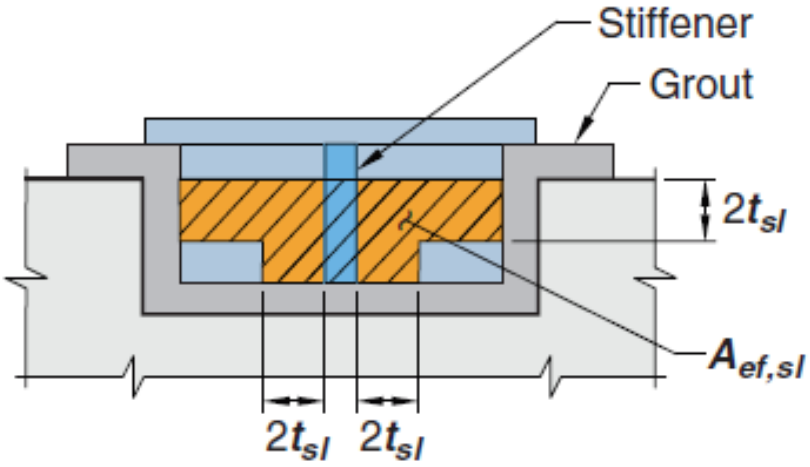
- $\phi V_{brg,sl} \geq V_u$
- Where $\phi = 0.65$



Source: Peter Carrato



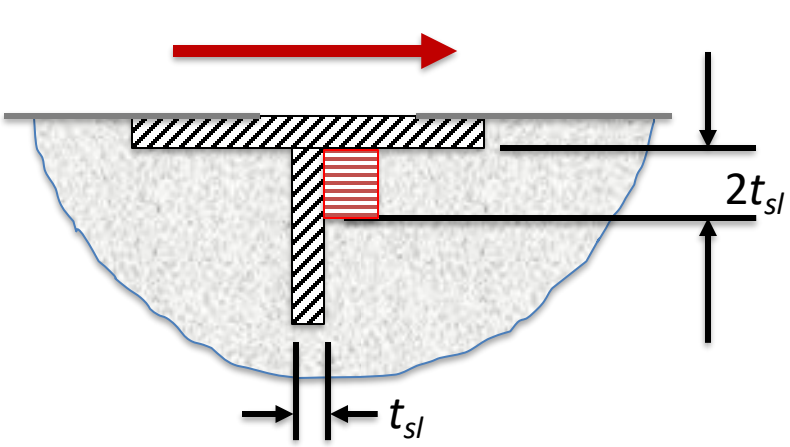
Bearing Strength (17.11.2)

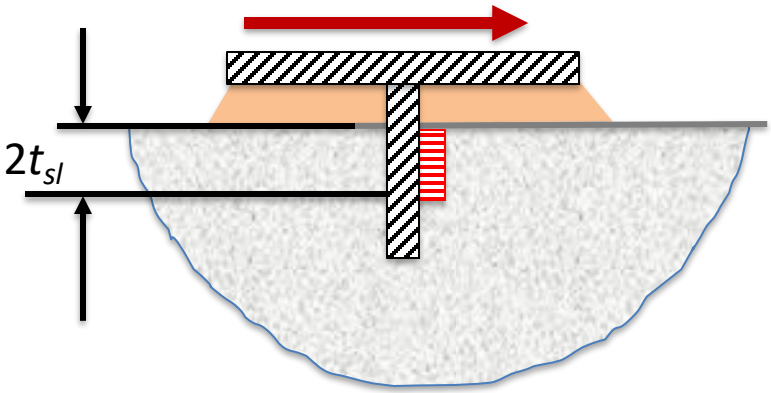


- Bearing strength:

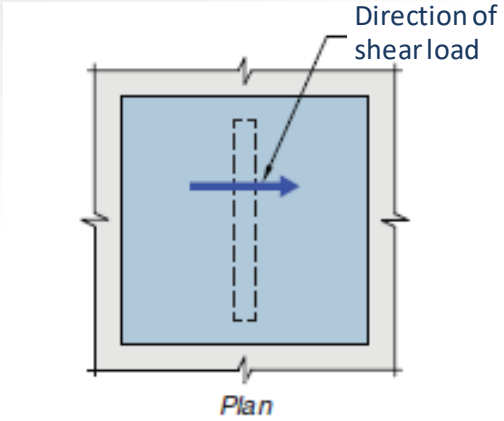
$$V_{brg,sl} = 1.7 f_c' A_{ef,sl} \Psi_{brg,sl}$$

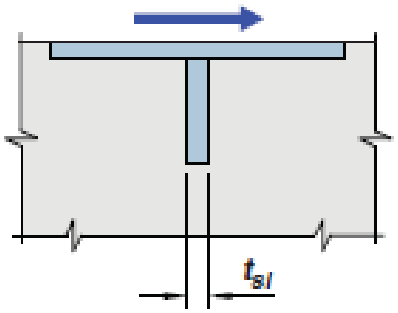
- $A_{ef,sl}$ is the surface perpendicular to the applied shear:



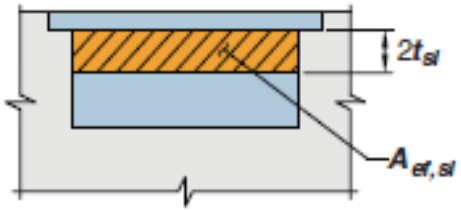


Bearing Area



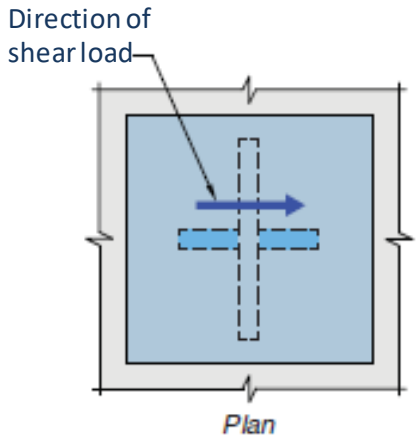


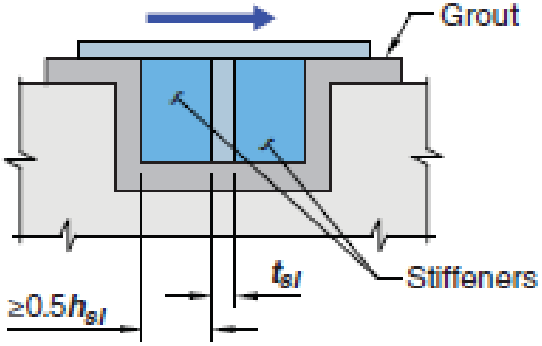
Elevation parallel to load



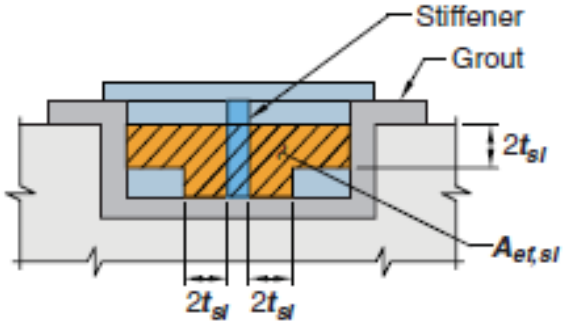
Elevation perpendicular to load

(a) Shear lug without stiffeners





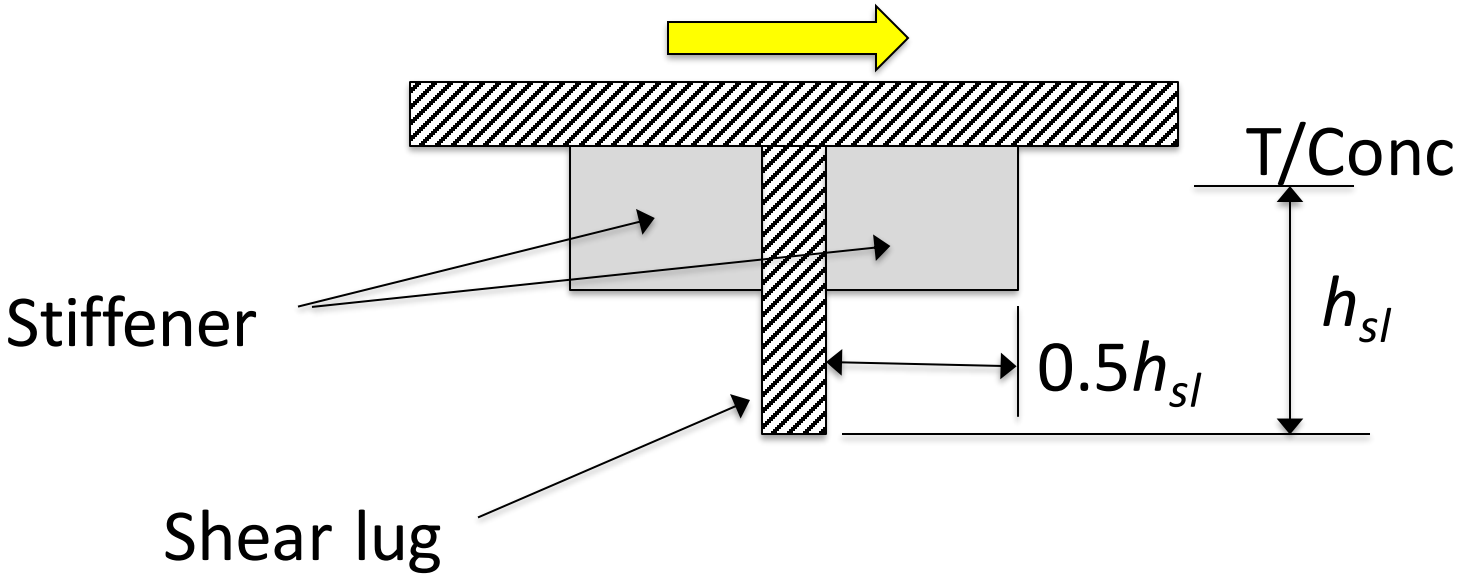
Elevation parallel to load



Elevation perpendicular to load

Stiffeners

- **17.11.2.3 - If used, the length of shear lug stiffeners in the direction of the shear load shall not be less than $0.5h_{sl}$**



17.11.2.2 – Bearing factor

$$V_{brg,sl} = 1.7 f_c' A_{ef,sl} \Psi_{brg,sl}$$

Tension load

- $\Psi_{brg,sl} = 1 + P_u/(nN_{sa}) \leq 1.0$
- P_u – negative for tension
- n – number of anchors in tension
- N_{sa} – Nominal tension strength of a single anchor

No applied axial load: $\Psi_{brg,st} = 1$

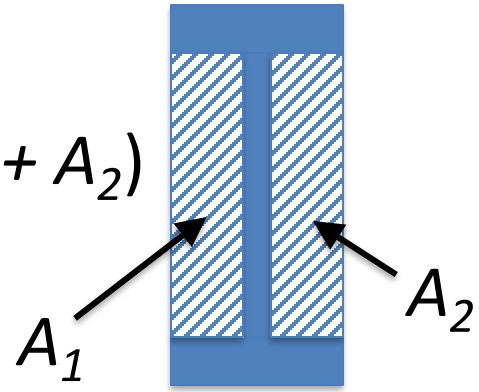
Compression load: $\Psi_{brg,sl} = 1 + 4P_u/(A_{bp}f_c') \leq 2.0$

- P_u – positive for compression

17.11.2.4 – Bearing for Multiple Shear Lugs

- If $\tau \leq 0.2 f'_c$, use bearing from both lugs

$$\tau = V_u/(A_1 + A_2)$$

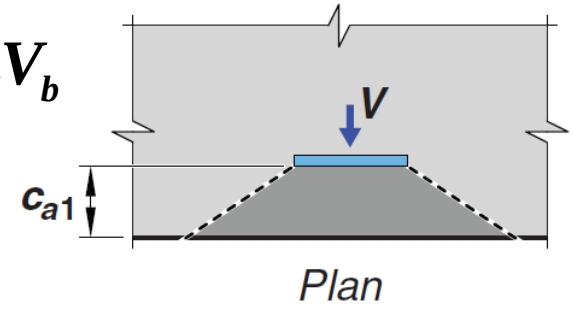




17.11.3 – Concrete breakout strength of shear lugs

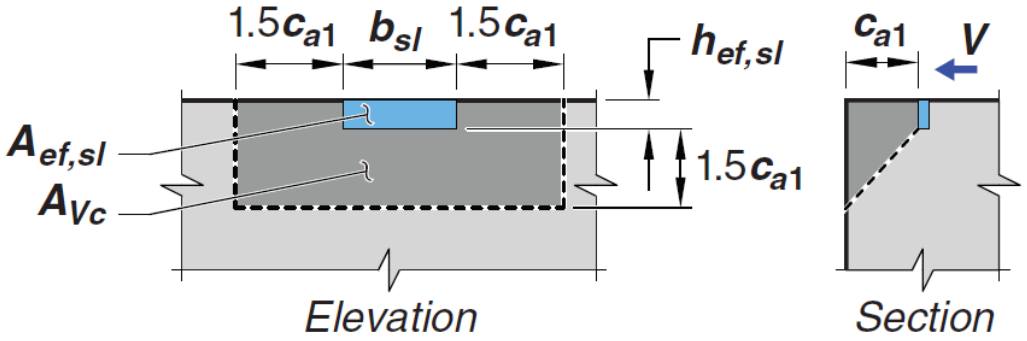
- **Nominal concrete breakout strength of a shear lug**
  - Use Anchor provisions of 17.7.2

$$V_{cb,sl} = \frac{A_{Vc}}{A_{Vco}} \psi_{ed,V} \psi_{c,V} \psi_{h,V} V_b$$



- **Where:**

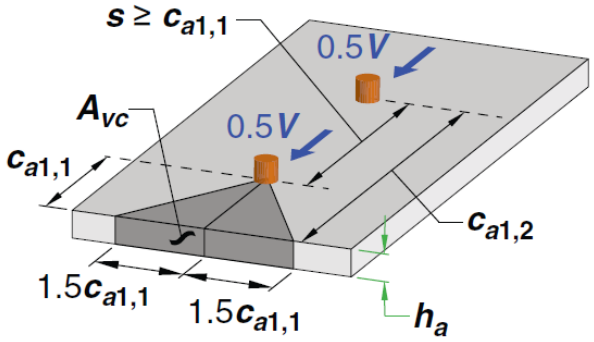
$$V_b = 9\lambda_a \sqrt{f_c'} (c_{a1})^{1.5}$$



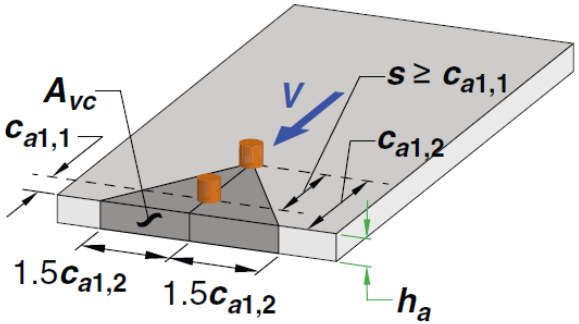
17.11.3.4 – Breakout for Multiple Shear Lugs

- Determine for each potential breakout surface
- Commentary directs to Fig. R17.7.2.1b

If $h_a < 1.5c_{a1}$
$A_{vc} = 2(1.5c_{a1,1})h_a$



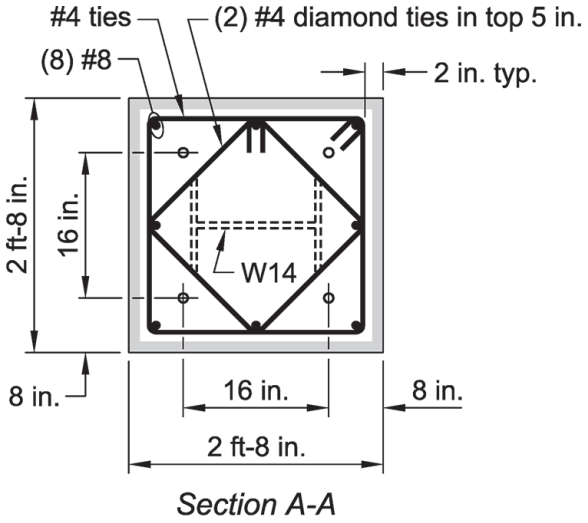
If $h_a < 1.5c_{a1}$
$A_{vc} = 2(1.5c_{a1,2})h_a$

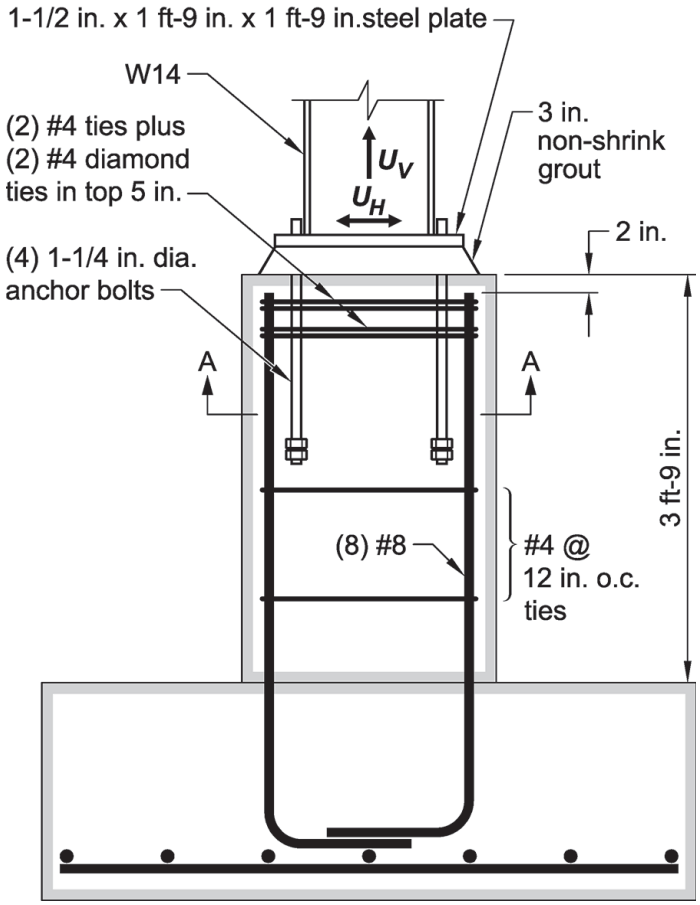


Shear Lug Example

- Reinforced Concrete Design Manual
- Anchorage example 20
- See handout

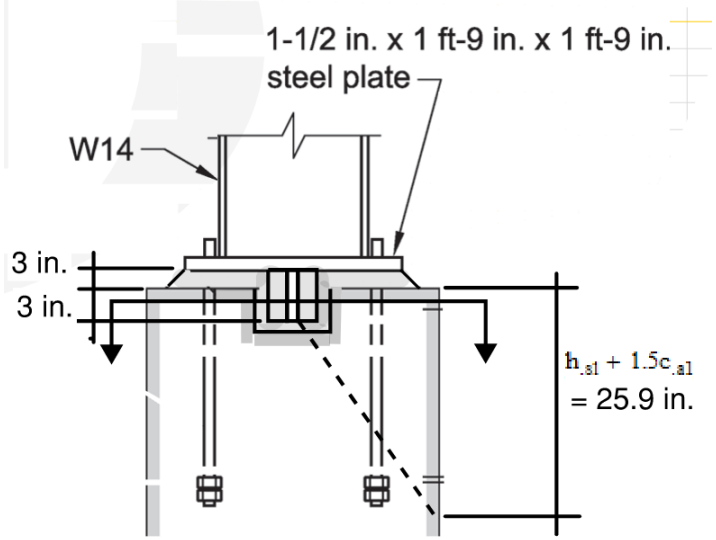
D_V = 60 Kips
L_V = 75 Kips
W_V = ±170 Kips
D_H = ± 8 Kips
L_H = ± 9 Kips
W_H = ±12 Kips

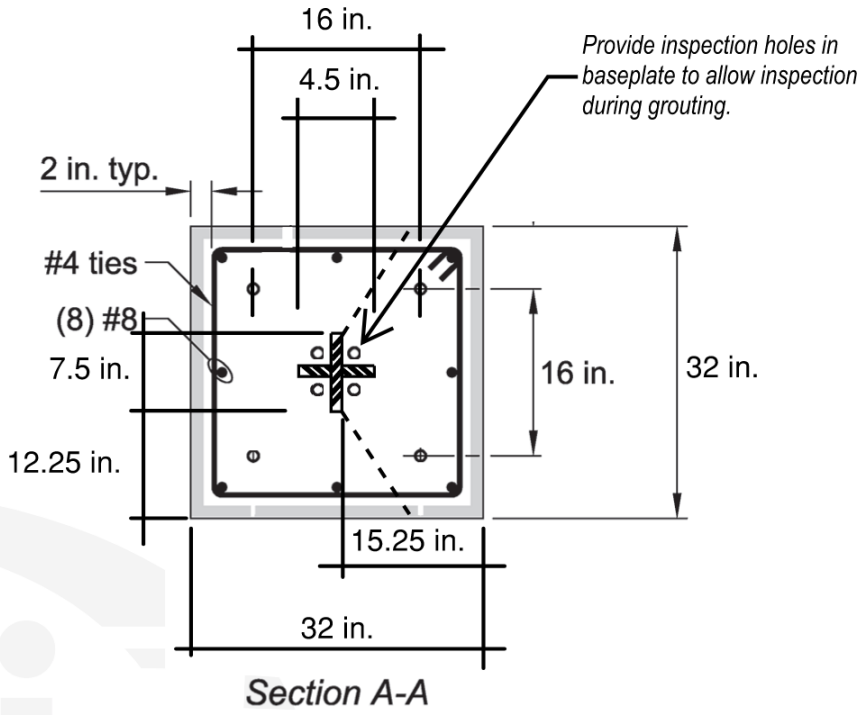




Shear Lug Example

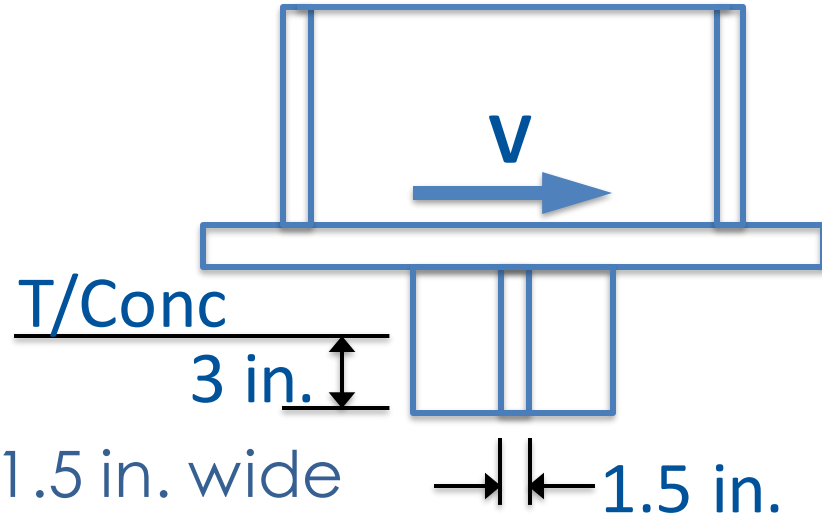
- **Can we replace upper ties with shear lug?**
  - Remove shear from anchor rod design
  - May reduce bolt size/length
  - Simplify design





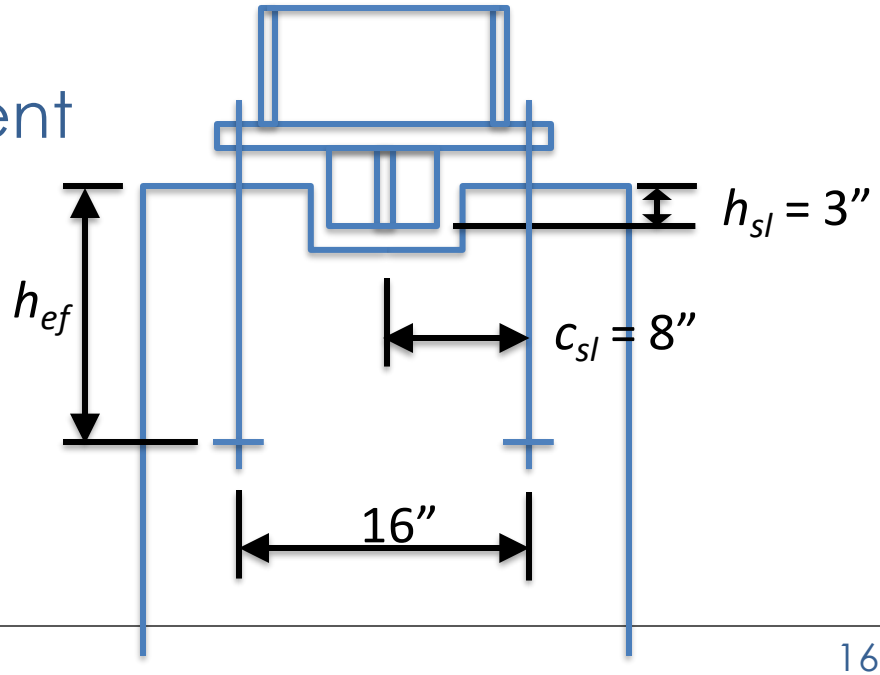
Size Shear Lug

- ***Size shear lug so entire lug is effective***
  - t_{sl} = 1.5 in.
  - Width = 1.5 in. + 4(1.5 in.)
    = 7.5 in.
  - Depth = 3 in. + 3 in.
    = 6 in.
  - Stiffeners at least 0.5 h_{sl} or 1.5 in. wide



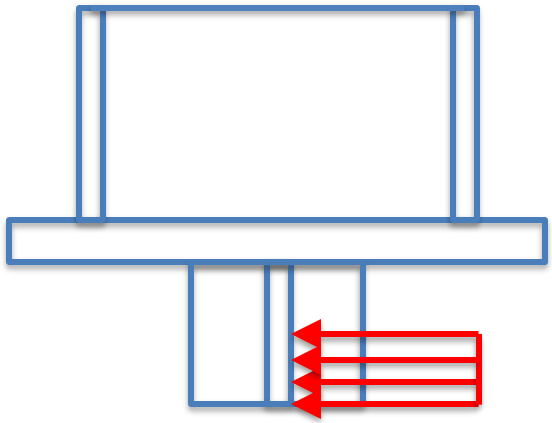
Shear Lug Example

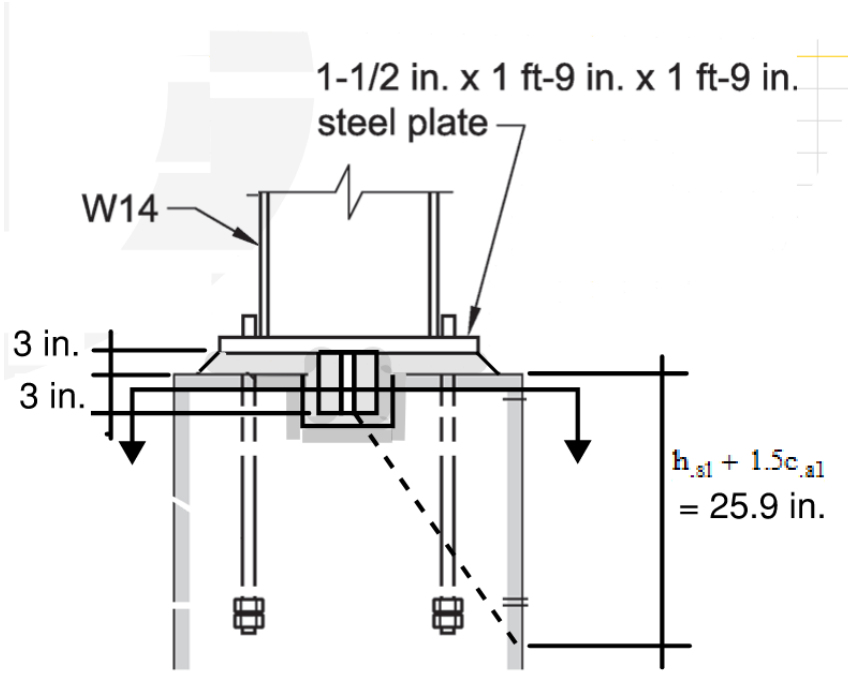
- **Check anchor rod depth (only required if attachment has tension)**
  - $h_{ef}/h_{sl} \geq 2.5 \rightarrow h_{ef} = 2.5\ (3\text{ in.}) = 7.5\text{ in.}$
  - $h_{ef}/c_{sl} \geq 2.5 \rightarrow h_{ef} = 2.5\ (8\text{ in.}) = 20\text{ in.}$ <= **controls**
  - Increase rod embedment from 18 in. to 20 in.



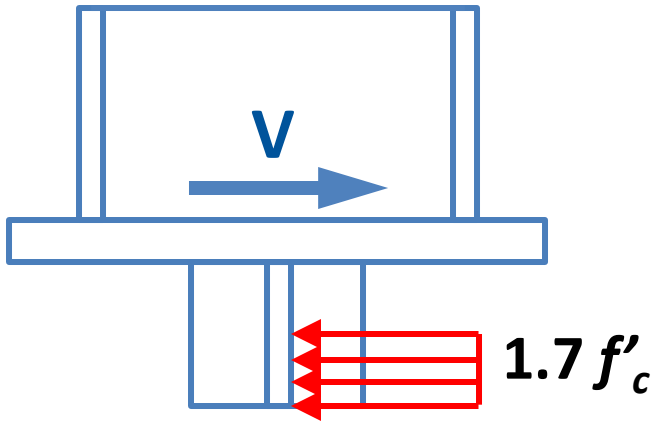
Strength Checks

- $V_{ua,g} \leq \phi V_{brg,sl}$ (bearing)
  $\leq \phi V_{cb,sl}$ (concrete breakout)
- $\phi = 0.65$





Bearing Strength Check



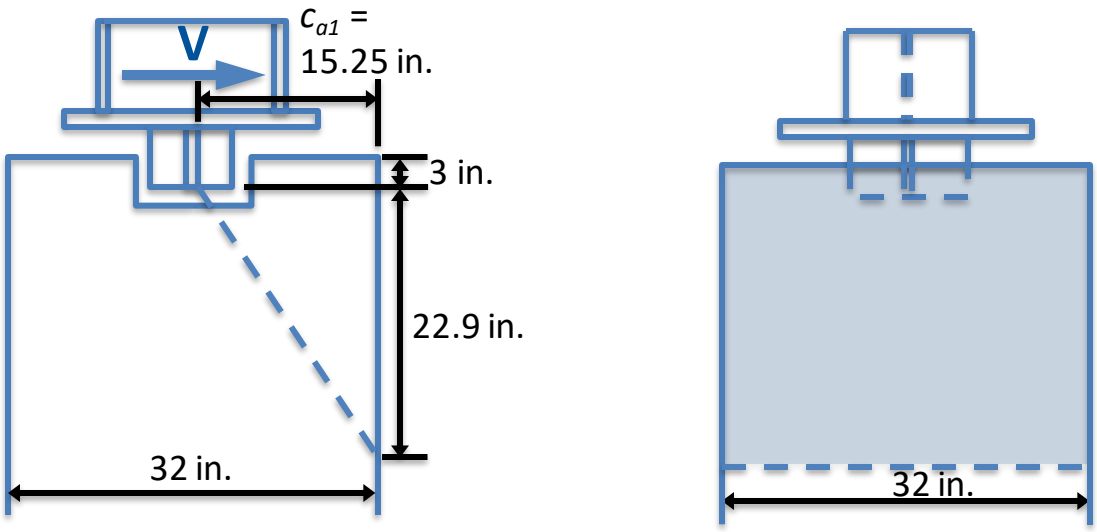
- $V_{ua,g} \leq \phi V_{brg,sl}$ **(bearing)**
  - $V_{ua,g}$ = 30 kip
  - $V_{brg,sl} = 1.7\, f'_c\, A_{ef,sl}\, \Psi_{brg,sl}$

- **For tension on attachment, bearing is reduced**
  - $\Psi_{brg,sl} = 1+P_u/(nN_{sa})$
  - $= 1+(-116 \text{ kip})/(4 \text{ rods}(72.7 \text{ kip/rod})) = 0.601$
  - $V_{brg,sl} = 1.7\,(4500 \text{ psi})(7.5 \text{ in.})(3 \text{ in.})(0.601) = 103$ kip

- $\phi V_{brg,sl}$ **= 0.65 (103 kip) = 67 kip > 30 kip OK**

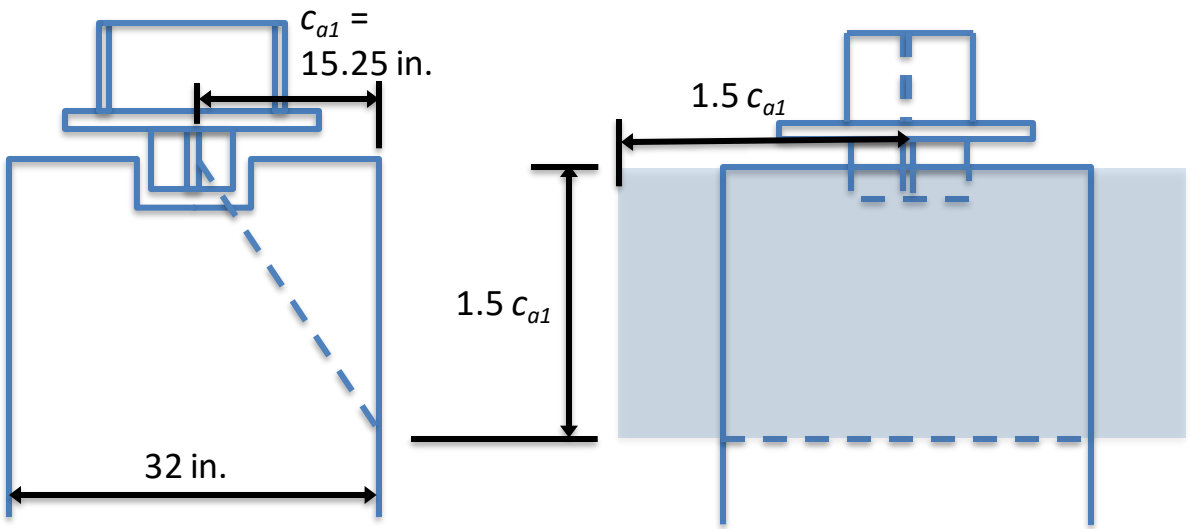
Concrete Breakout Strength Check

- $V_{ua,g} \leq \phi V_{cb,sl}$ (concrete breakout)
- $V_{cb,sl} = (A_{Vc}/A_{Vc0})\, \Psi_{ed,V}\, \Psi_{c,V}\, \Psi_{h,V}\, V_b$
  - A_{Vc} = [3" + 1.5 (32" -1.5")/2](32")-(3")(7.5")
    = 805 in.^{2}



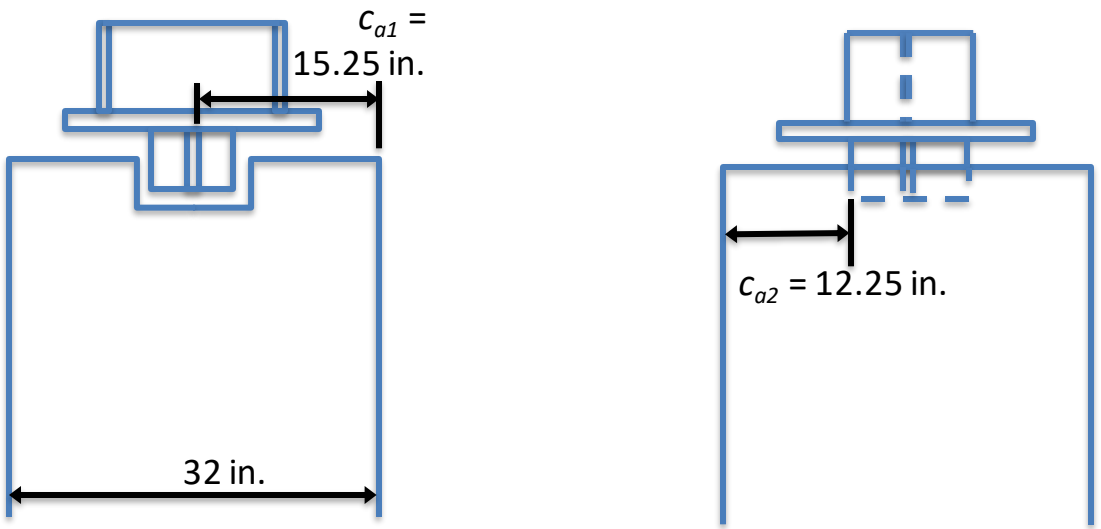
Concrete Breakout Strength Check

- $V_{cb,sl} = (A_{Vc}/A_{Vc0})\,\Psi_{ed,V}\,\Psi_{c,V}\,\Psi_{h,V}\,V_b$
  - $A_{Vc0} = 4.5\,c_{a1}^2 = 4.5(15.25\text{“})^2 = 1047$ in.^{2}



Concrete Breakout Strength Check

- $V_{cb,sl} = (A_{Vc}/A_{Vc0})\,\Psi_{ed,V}\,\Psi_{c,V}\,\Psi_{h,V}\,V_b$
  - $\Psi_{ed,V}$ = edge effect modification factor
    $= 0.7 + 0.3c_{a2}/(1.5c_{a1})$
    $= 0.7+0.3(12.25")/(1.5(15.25"))=0.861$



Concrete Breakout Strength Check

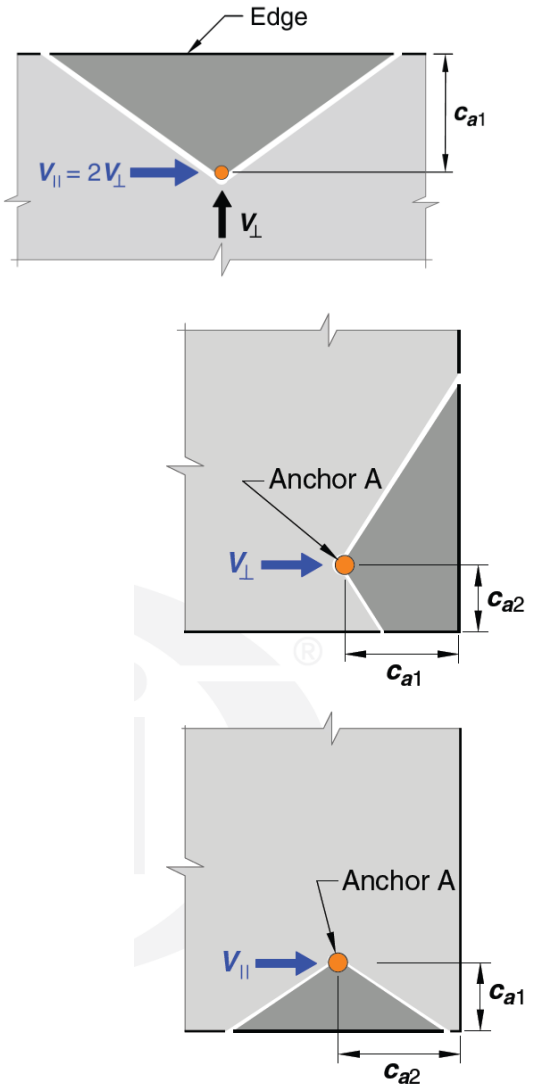
- $V_{cb,sl} = (A_{Vc}/A_{Vc0})\,\Psi_{ed,V}\,\Psi_{c,V}\,\Psi_{h,V}\,V_b$
  - $\Psi_{c,V}$ = concrete cracking modification factor
  - Assume cracking and No. 4 ties between lug and edge (see Table 17.7.2.5.1)
  - $\Psi_{c,V} = 1.2$
  - $\Psi_{h,V}$ = member thickness modification factor
    $= 1.0$ (depth $> 1.5\,c_{a1}$)
  - $V_b = 9\lambda_a\sqrt{f'_c}(c_{a1})^{1.5}$
    $= 9(1)(\sqrt{4500}\text{ psi})(15.25'')^{1.5} = 36{,}000$ lb

Concrete Breakout Strength Check

- $V_{cb,sl} = (A_{Vc}/A_{Vc0})\,\Psi_{ed,V}\,\Psi_{c,V}\,\Psi_{h,V}\,V_b$
  $= (805\text{ in.}^2/1047\text{ in.}^2)(0.861)(1.2)(1.0)(36\text{ kip})$
  $= 28.6\text{ kip}$
- $\phi\,V_{cb,sl} = 0.65(28.6\text{ kip}) = 18.6\text{ kip} < 30\text{ kip} \leftarrow$ NG

Shear parallel to an edge or at a corner

- **Shear parallel to an edge**
  - 17.11.3.2 → 17.7.2.1(c)
- **Shear at a corner**
  - 17.11.3.3 → 17.7.2.1(d)



Summary

- **$\phi V_{cb,sl}$ = 18.6 kip < 30 kip ← anchor reinforcement required**
- **From example:**
  - all 4 rods resisting and supplementary reinforcement → ϕV_{cbg} = 29.4 kip
  - back 2 rods resisting and supplementary reinforcement → $\phi V_{cb,sl}$ = 21.7 kip
- **Shear lugs not helpful for breakout**
- **Helpful when shear in rods is controlling**

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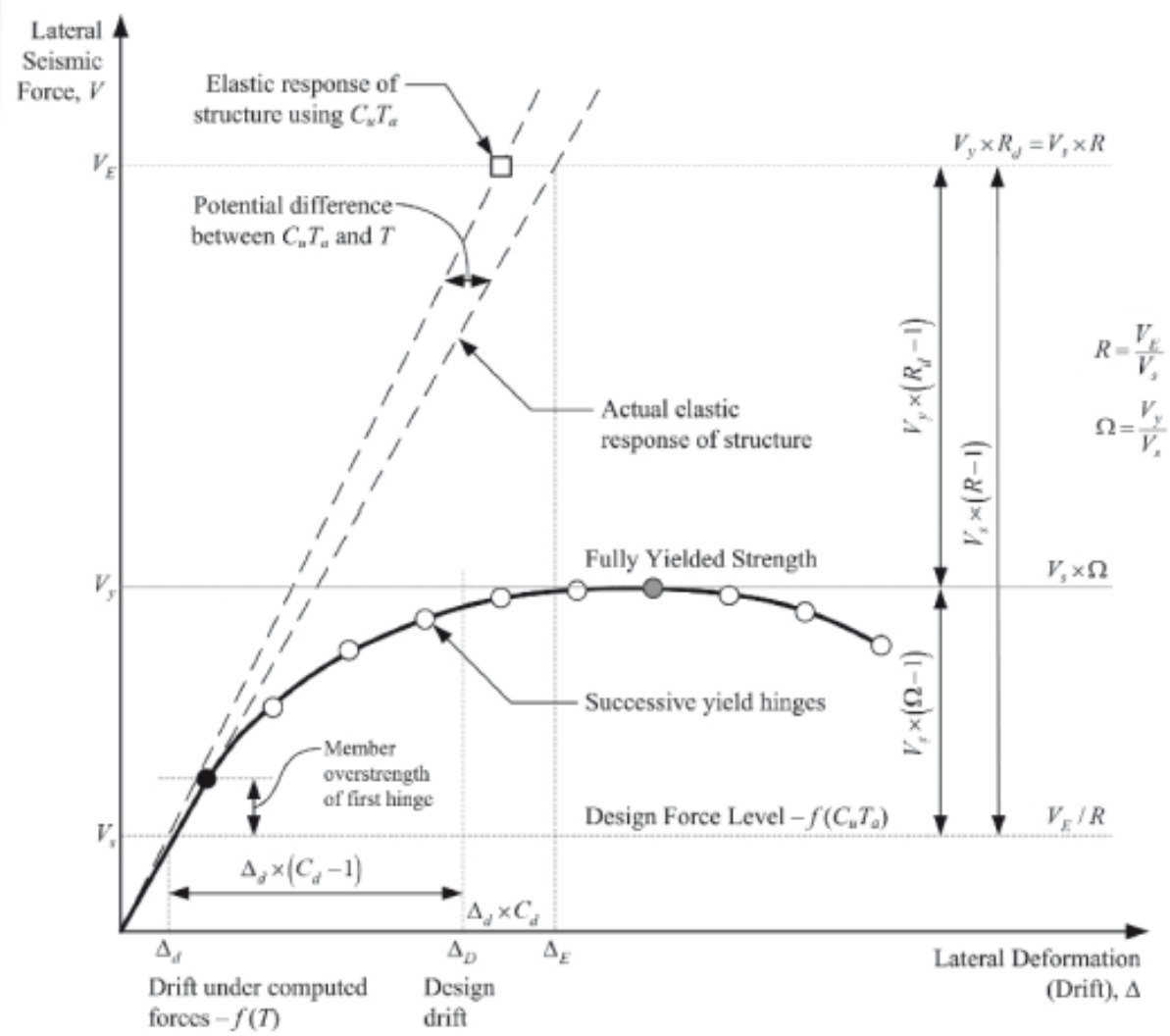
Seismic Design Philosophy

Seismic

- Both concrete and reinforcement are <u>permitted to respond in the inelastic range</u>
- This is consistent with the strength design approach adopted throughout the Code

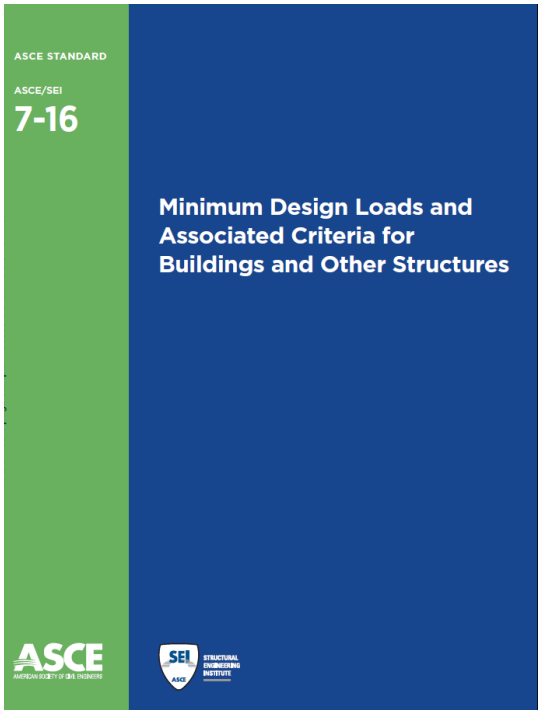


Seismic – Ω, C_d, and R Factors (ASCE 7)



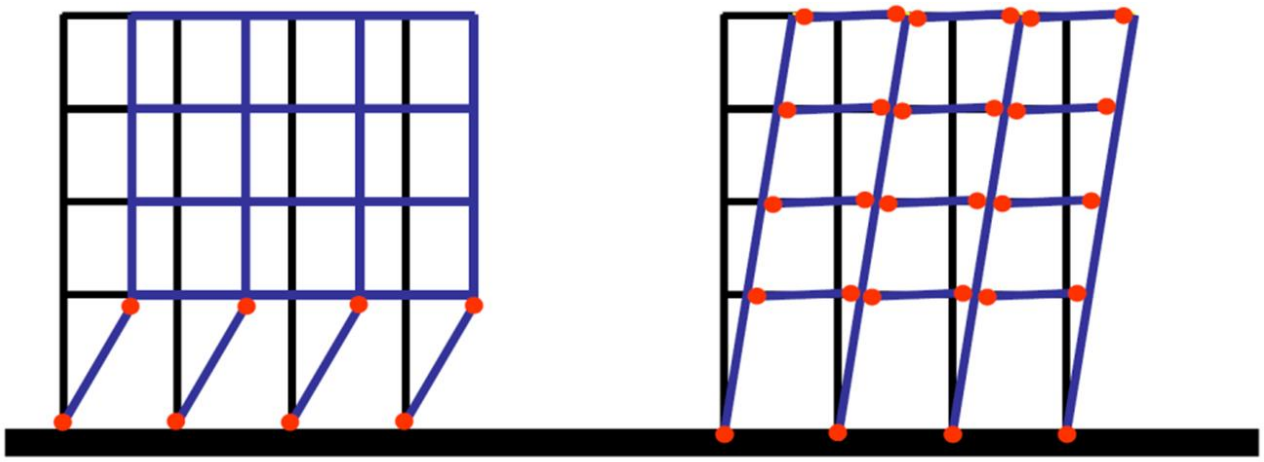
Seismic – Parameters

Parameter in ASCE 7-16 Table 12.2-1	Example
Seismic Force Resisting System	Special reinforced concrete shear walls (building frame system)
ASCE 7 Section Where Detailing Requirements Are Specified	ASCE 7 Section 14.2 "Concrete"
Response Modification Coefficient, ***R***	6
Overstrength Factor, Ω_0	2.5
Deflection Amplification Factor, C_d	5
Structural System Limitations, Including Structural Height Limits	SDC B No limit SDC C No limit SDC D 160 ft SDC E 160 ft SDC F 100 ft



Seismic

- Controlled inelastic action is permitted at pre-determined locations, called plastic hinges
- Typical plastic hinge locations are at the ends of beams in moment frames, and at the bases of shear walls



Story Mechanism Sway Mechanism

Seismic

- Prescriptive rules for detailing of reinforcement are enforced, creating robust plastic hinges
- Plastic hinging reduces the stiffness of the structure, which lengthens the period; and plastic hinges dissipate earthquake energy



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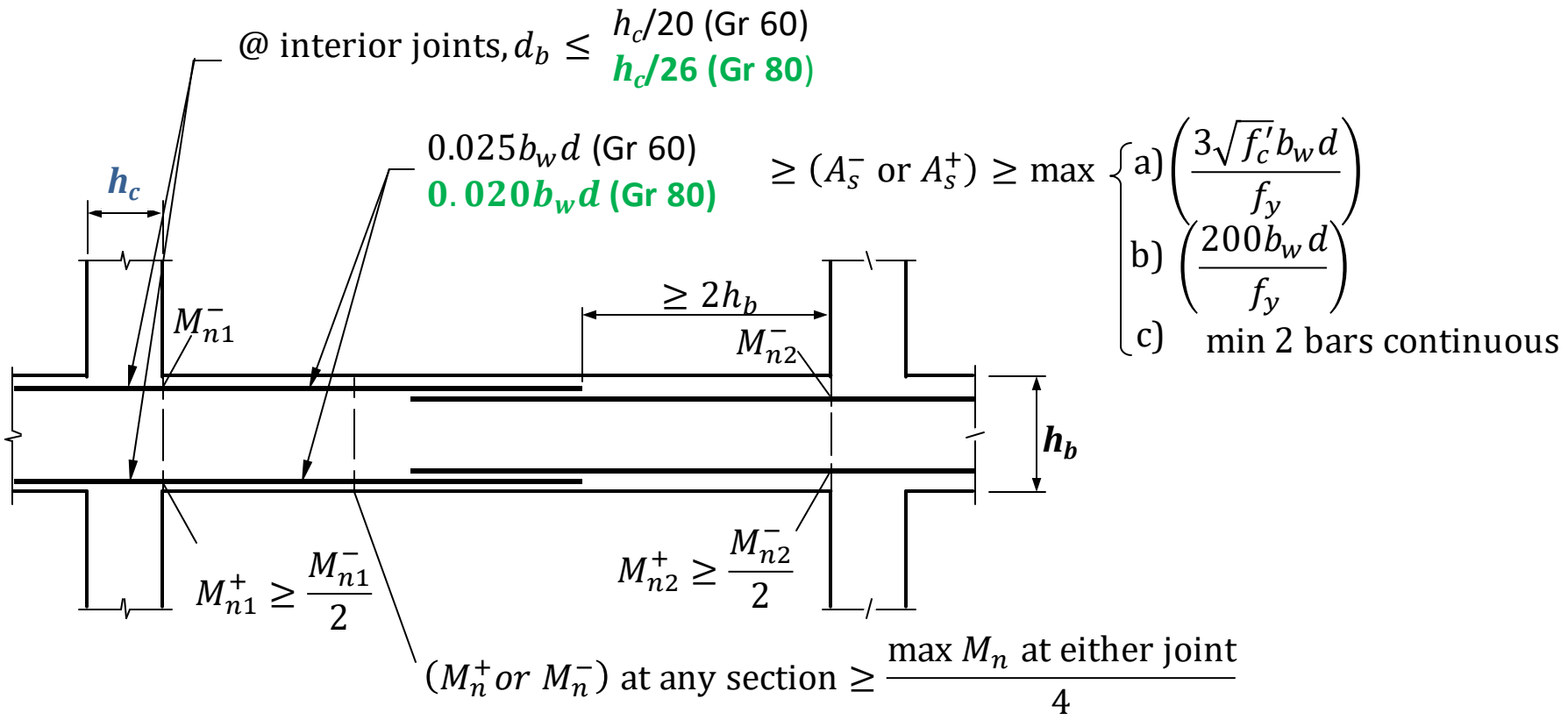
Changes to the Concrete Design Standard



Special Moment Frames

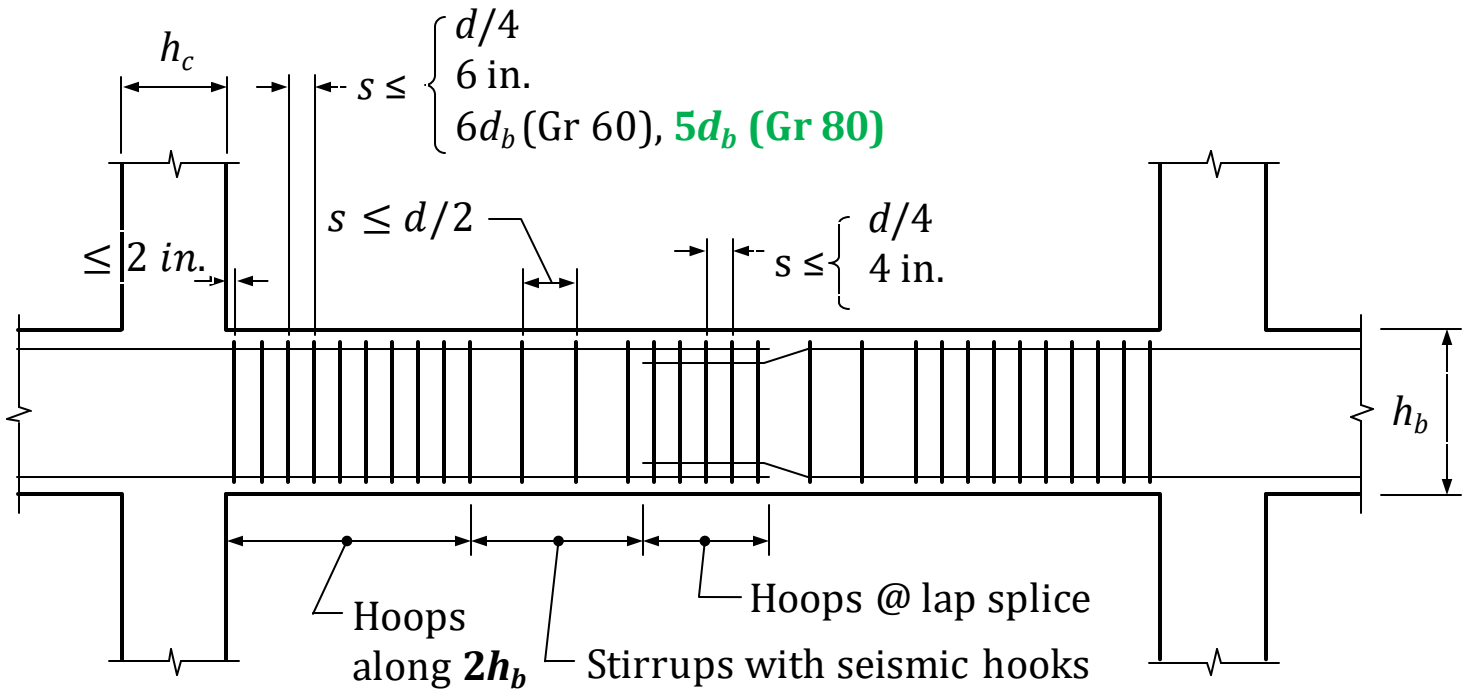
18.6.3.1 and 18.8.2.3—Special moment frame beams (and joints)

- **Longitudinal Reinforcement**



18.6.4.4—Special moment frame beams

- **Transverse reinforcement**

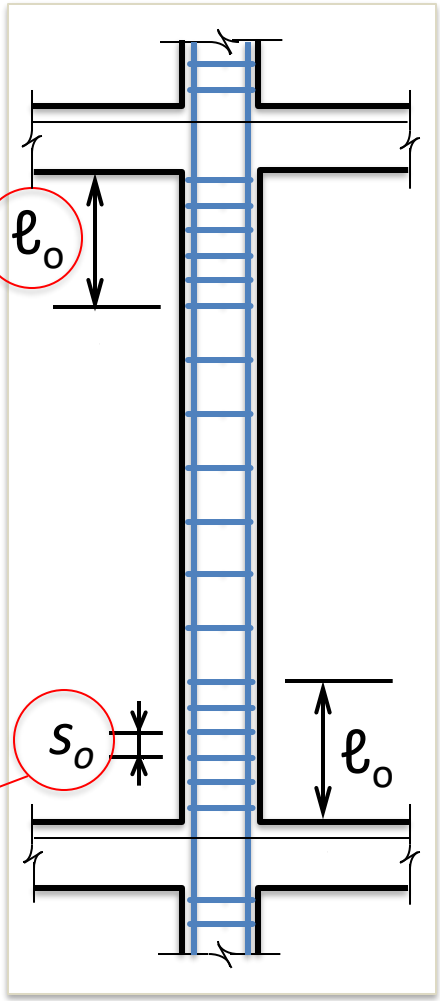


18.4.3.3—Columns in intermediate moment frames

- Hoops or spirals required
- First hoop at $s_o/2$ from the joint face

$$\ell_o \geq \begin{cases} \ell_u/6 \text{ clear span} \\ [c_1, c_2]_{max} \\ 18 \text{ in.} \end{cases}$$

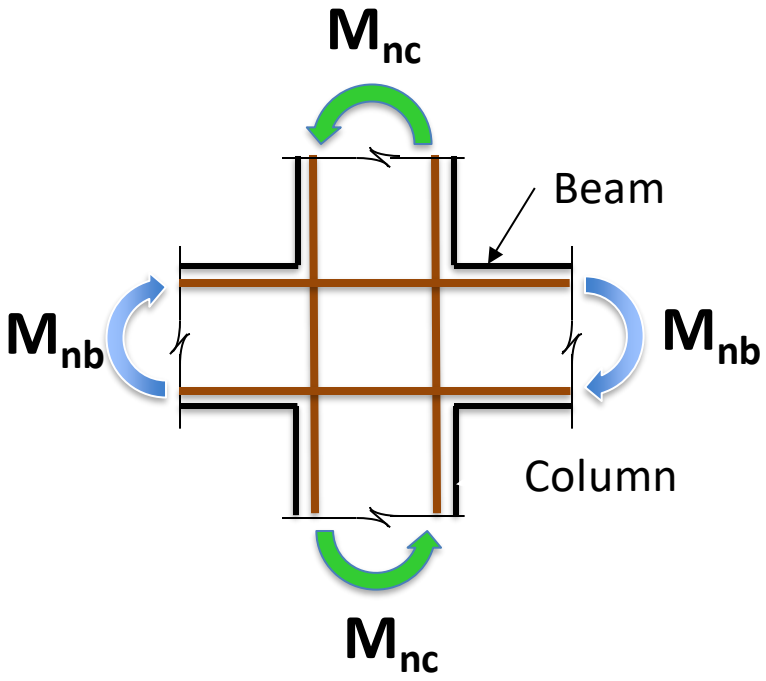
$$s_o \leq \begin{cases} 8d_b \text{ (Gr 60) and 8 in.} \\ 6d_b \text{ (Gr 80) and 6 in.} \\ 1/2[c_1, c_2]_{min} \end{cases}$$

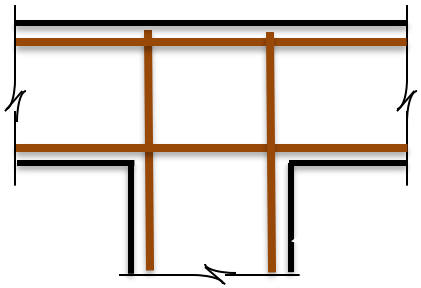


18.7.2, 18.7.3—Columns of SMF

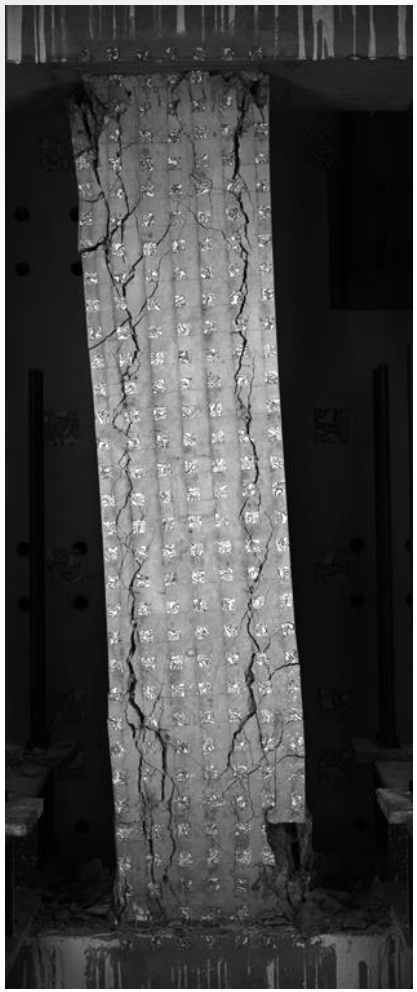
Strong Column/Weak Beam

- **Column dimensional limits, 18.7.2**
  - Smallest dimension ≥ 12 in.
  - Short side/long side ≥ 0.4
- **Flexural strength check, 18.7.3.2**
  - $\sum M_{nc} \geq (6/5)\sum M_{nb}$,
  - Exception, 18.7.3.1
    - Ignore check at top story where $\boldsymbol{P_u \leq 0.1 A_g f'_c}$





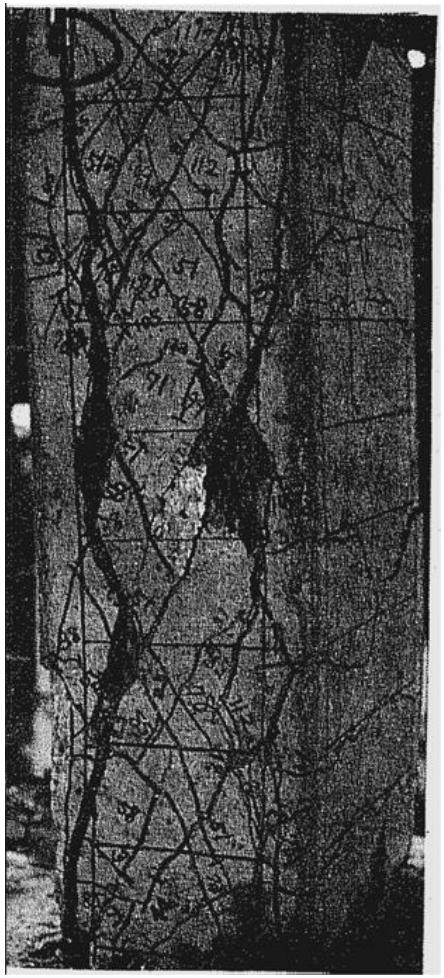
18.7.4.3—Bond splitting failure in columns



Sokoli and Ghannoum (2016)

Splitting can be controlled by restricting the longitudinal bar size to meet

$$1.25\ell_d \leq \ell_u/2$$



Woodward and Jirsa (1984)
Umehara and Jirsa (1982)

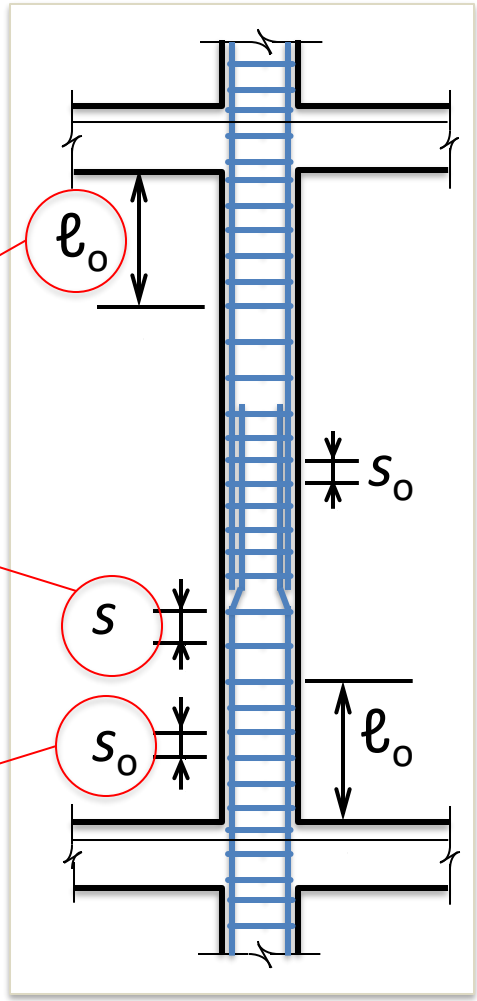
18.7.5.3 and 18.7.5.5—Columns in special moment frames

- First hoop at $s_o/2$ from the joint face

$$\ell_o \geq \begin{cases} \ell_u/6 \text{ clear span} \\ [c_1, c_2]_{max} \\ 18 \text{ in.} \end{cases}$$

$$s \leq \begin{cases} 6d_{b,min} \text{ (Gr 60)}, \ 5d_{b,min} \text{ (Gr 80)} \\ 6 \text{ in.} \end{cases}$$

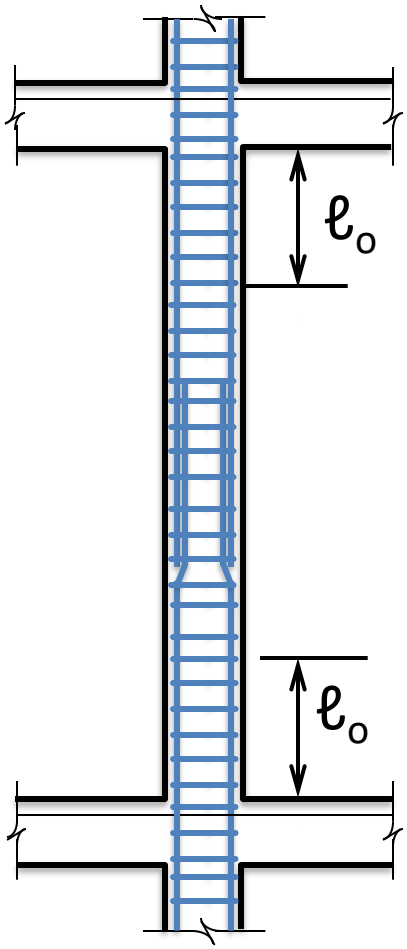
$$s_o \leq \begin{cases} 6d_{b,min} \text{ (Gr 60)}, \ 5d_{b,min} \text{ (Gr 80)} \\ \tfrac{1}{4}[c_1, c_2]_{min} \\ 4 + \left(\frac{14 - h_x}{3}\right), \leq 6 \text{ in.}; \geq 4 \text{ in.} \end{cases}$$



18.14.3.2—Nonparticipating columns

Clarification

- **Transverse spacing over full length is the lesser of**
  - $6d_b$ of the smallest long. bar
  - 6 in.
- **Transverse detailing along ℓ_o is according to 18.7.5.2 (a) through (e)**
  - 18.7.5.2(f) is not required



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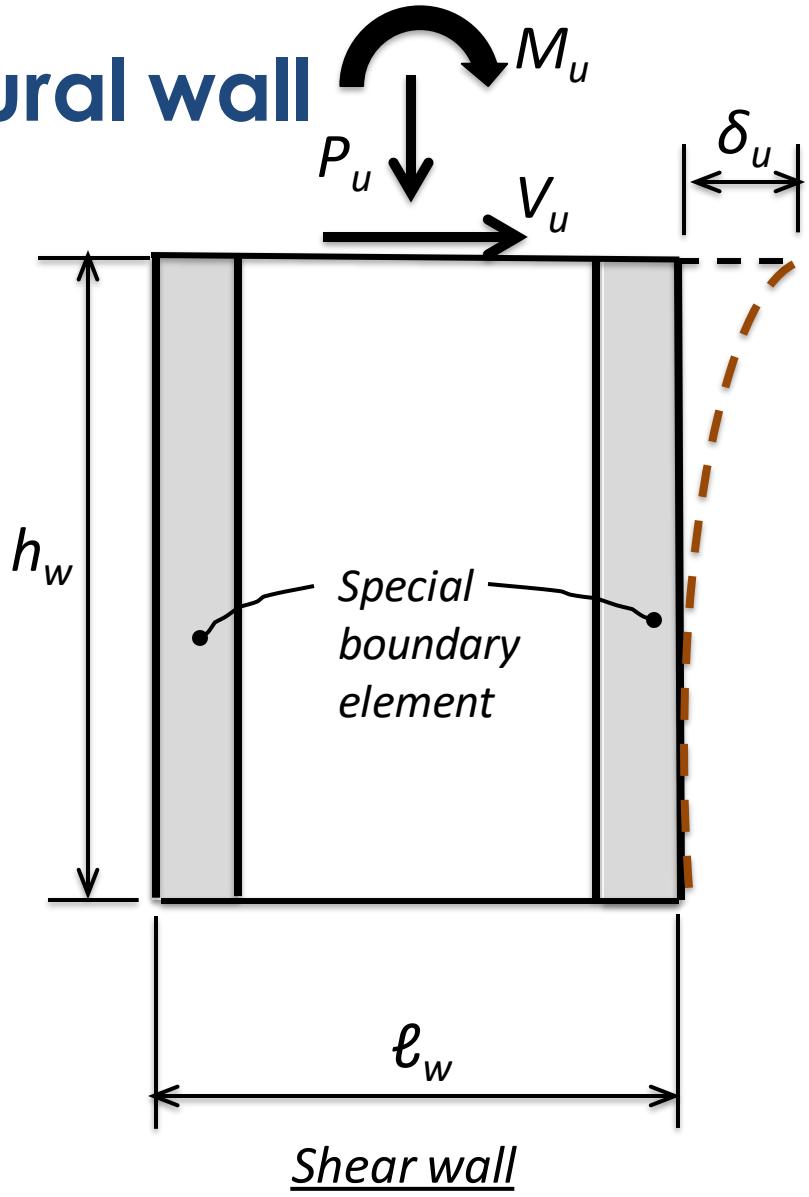
Changes to the Concrete Design Standard



Special Structural Walls

Ch. 18.10—Special structural wall

- Cutoff of longitudinal bars in special boundary elements
- Reinforcement ratios at ends of walls
- Shear demand
- Drift capacity check
- Detailing in special boundary elements
- Ductile coupled walls

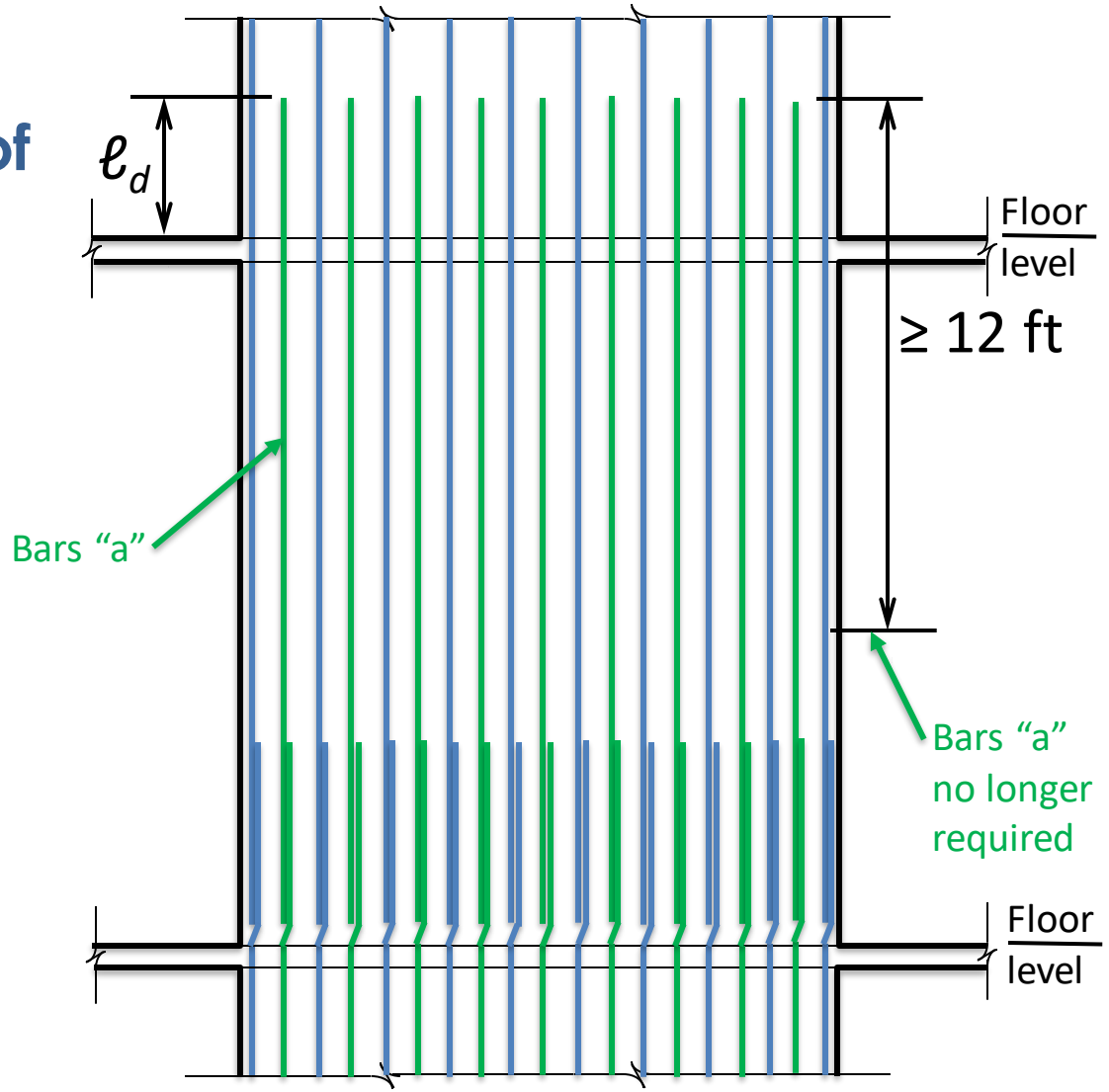


18.10.2.3(a)—Longitudinal bars

- **Previously,**
  - tension (vertical boundary) reinforcement in special structural walls to extend $0.8\ell_w$ beyond the point at which it is no longer required to resist flexure
- **Overly conservative**
  - This was an approximation of d
  - Similar to beams which extend d, $12d_b$ and $\ell_n/16$
  - Actual behavior is different

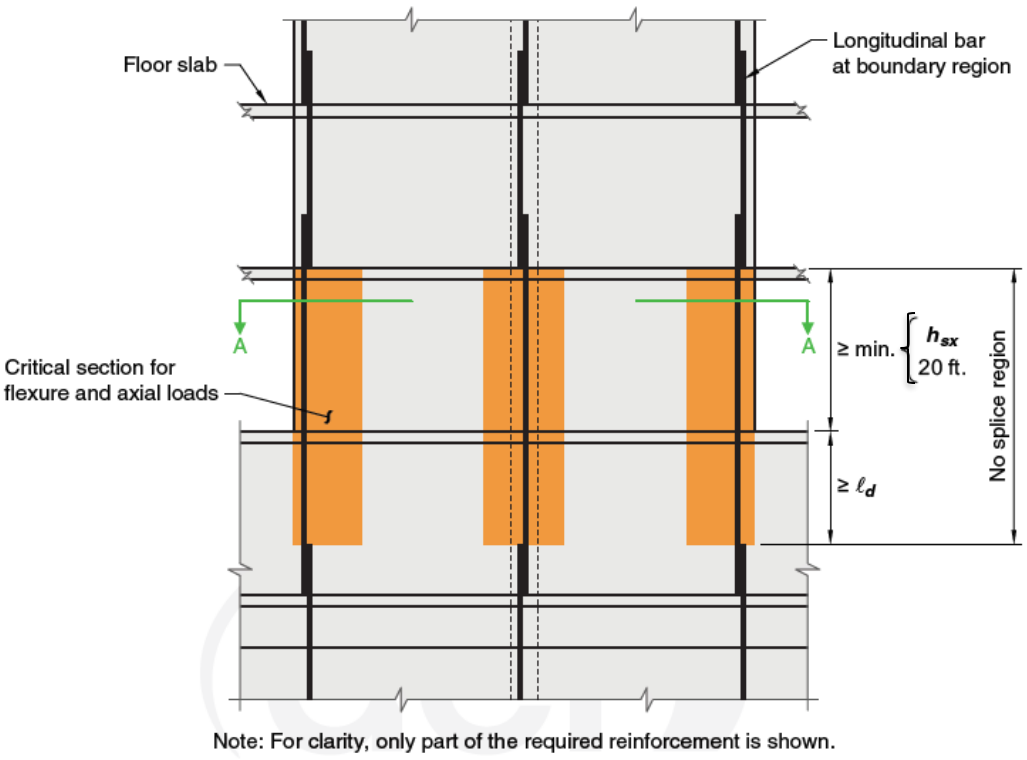
18.10.2.3(a)—Longitudinal bars

(a) Except at the top of a wall, longitudinal reinforcement shall extend at least 12 ft above the point at which it is no longer required to resist flexure but need not extend more than ℓ_d above the next floor level.

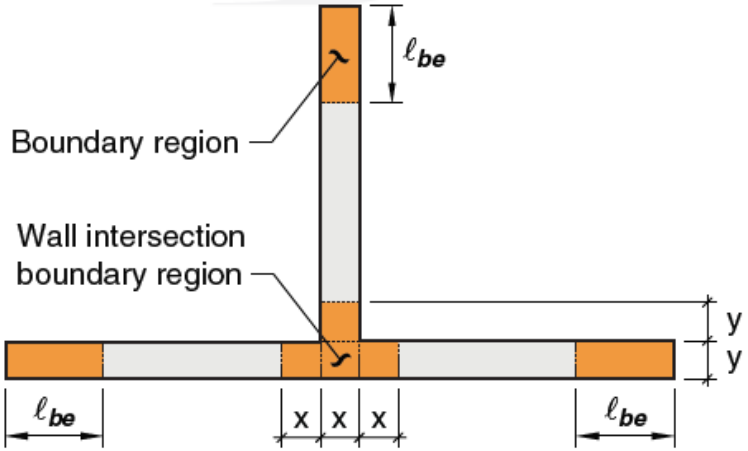


18.10.2.3(c)—Longitudinal bars

- Lap splices not permitted over h_{sx} above (20 ft, max) and ℓ_d below critical sections



Note: For clarity, only part of the required reinforcement is shown.

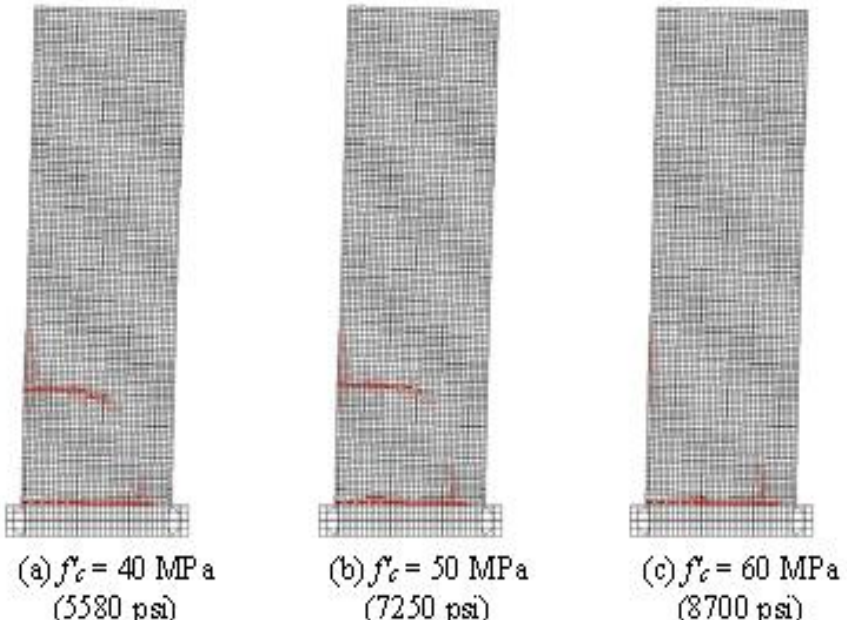


(b) Section A-A

18.10.2.4—Longitudinal reinforcement ratio at ends of walls

$h_w/\ell_w \geq 2.0$

- Failures in Chile and New Zealand
- 1 or 2 large cracks
- Minor secondary cracks



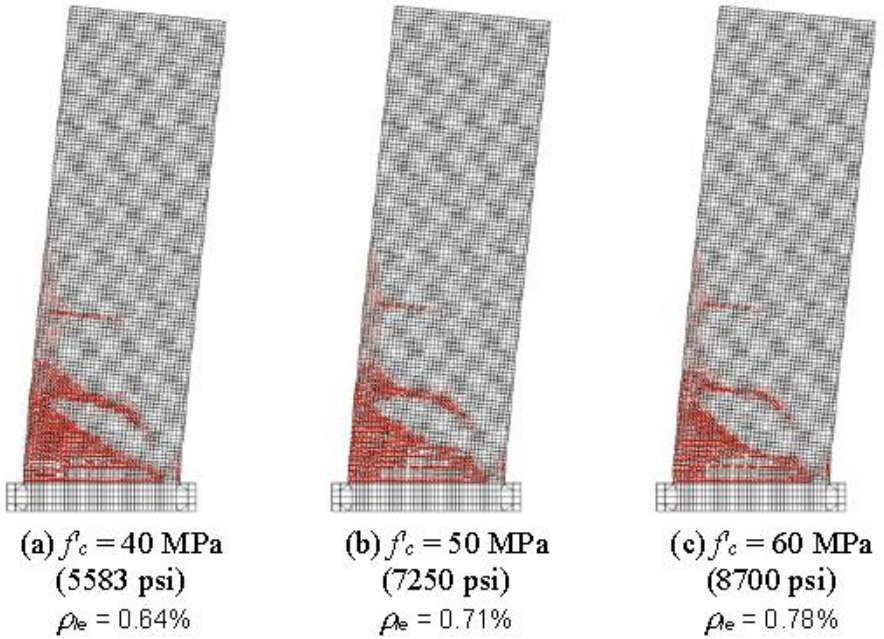
Crack patterns for walls with fixed minimum longitudinal reinforcement content of 0.25% (Lu et al. 2017)

18.10.2.4—Longitudinal reinforcement ratio at ends of walls

New ratio

$$\rho = \frac{6\sqrt{f_c'}}{f_y}$$

- Many well distributed cracks
- Flexure yielding over length

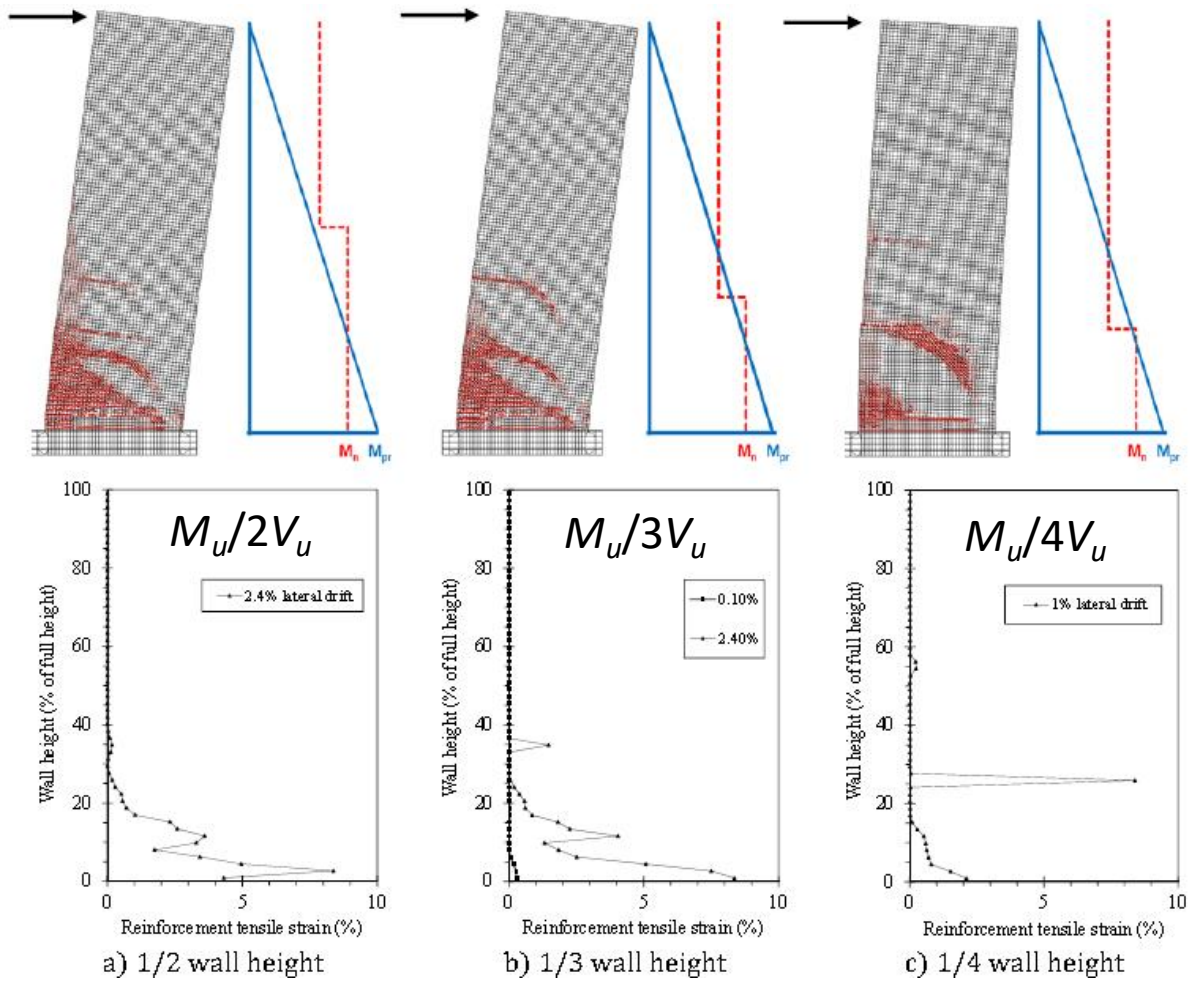


Crack patterns for walls with ρ according to equation (Lu et al. 2017)

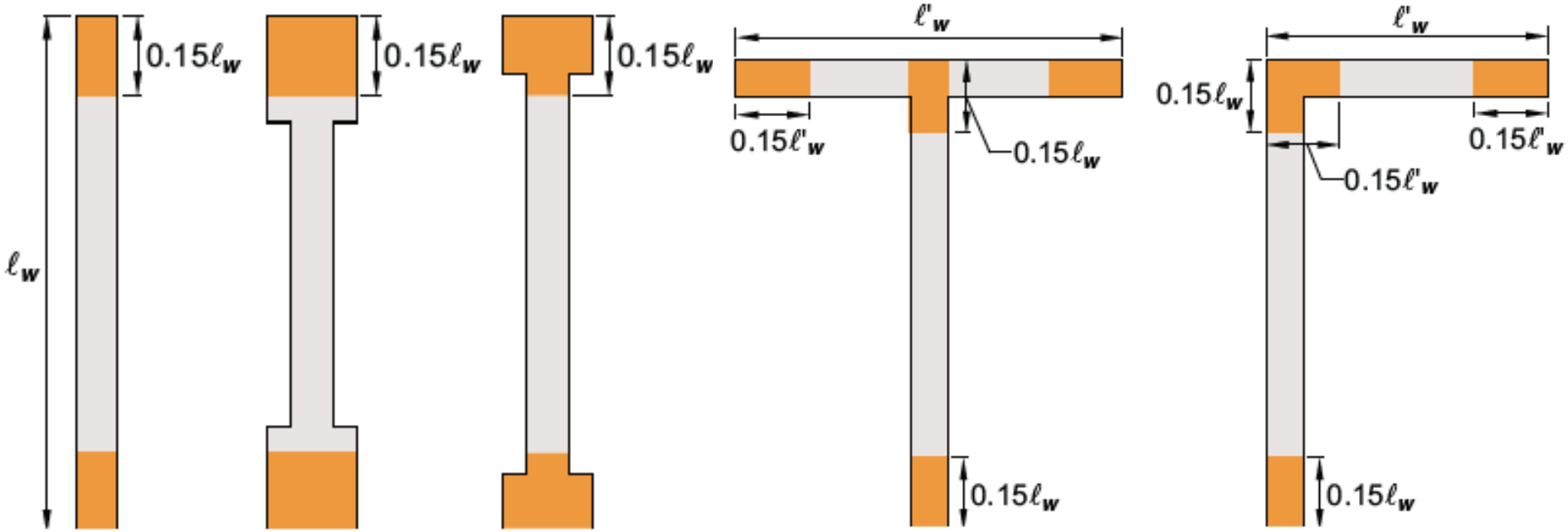
18.10.2.4—Longitudinal reinforcement ratio at ends of walls

Bar Cutoff

- $M_u/2V_u$ similar to wall with full reinforcement
- $M_u/3V_u$ good distribution
- $M_u/4V_u$ significant strain above cut off



18.10.2.4—Longitudinal reinforcement ratio at ends of walls



18.10.2.4—Longitudinal reinforcement ratio at ends of walls

Walls or wall piers with $h_w/\ell_w \geq 2.0$ must satisfy:

a) Long. reinf. ratio within 0.15 ℓ_w and minimum

$$\rho = \frac{6\sqrt{f_c'}}{f_y}$$

b) Long. reinf. extends above and below critical section the greater of ℓ_w and $M_u/3V_u$

c) Max. 50% of reinf. terminated at one section

18.10.3—Shear amplification

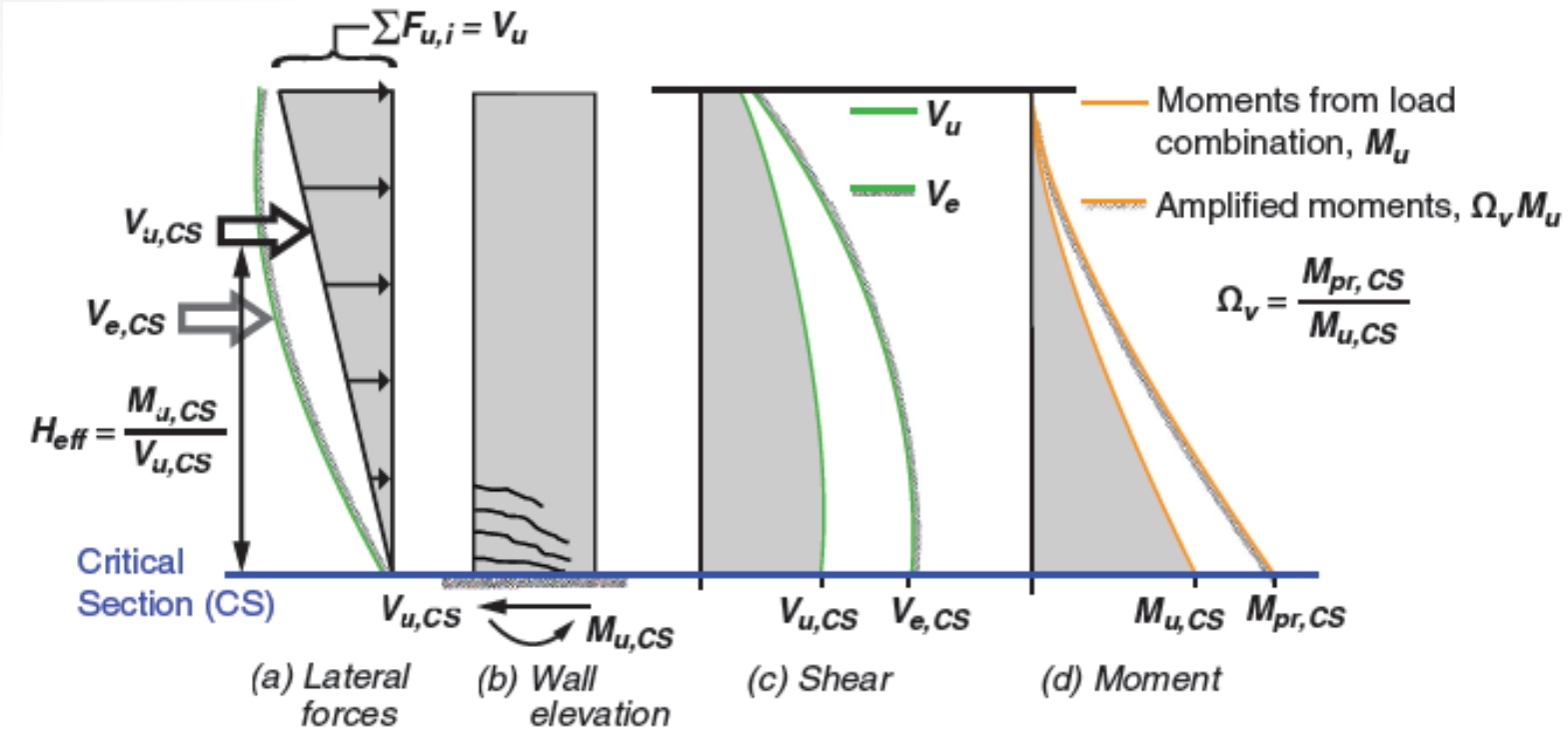


Fig. R18.10.3.1—Determination of shear demand for walls with $h_w/\ell_w \geq 2.0$ (Moehle et al 2011).

- Similar to approach in New Zealand Standard, NZS 3101

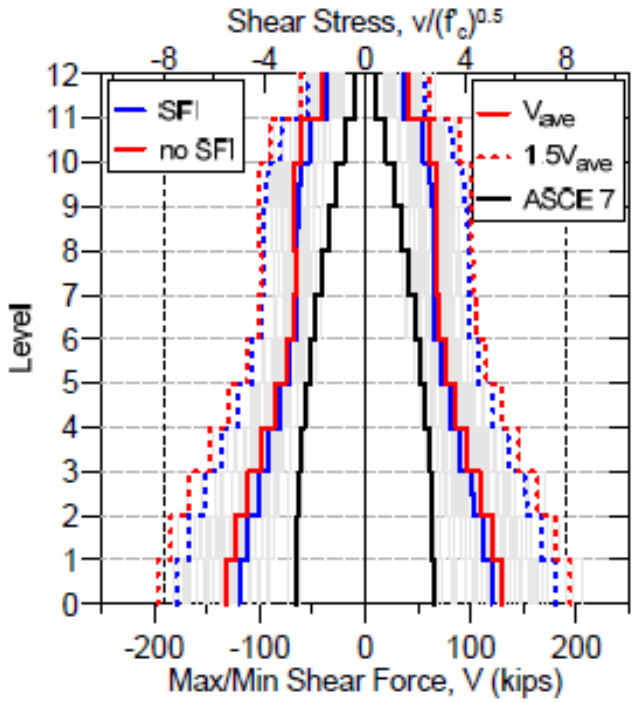
18.10.3—Shear amplification

18.10.3.1 The design shear force V_e shall be calculated by: $V_e = \Omega_v \omega_v V_u \leq 3V_u$

V_u = the shear force obtained from code lateral load analysis with factored load combinations

Ω_v = overstrength factor equal to the ratio of M_{pr}/M_u at the wall critical section.

ω_v = factor to account for dynamic shear amplification.



Gogus and Wallace, 2015

18.10.3—Shear amplification

18.10.3.1.2 – Calculation of Ω_v

Table 18.10.3.1.2—Overstrength factor Ω_v at critical section

Condition	Ω_v	
$h_{wcs}/\ell_w > 1.5$	Greater of	M_{pr}/M_u[1]
		1.5[2]
$h_{wcs}/\ell_w \leq 1.5$	1.0	

[1] For the load combination producing the largest value of Ω_v.

[2] Unless a more detailed analysis demonstrated a smaller value, but not less than 1.0.

18.10.3—Shear Amplification

18.10.3.1.3 – Calculation of ω_v

$h_{wcs}/\ell_w < 2.0 \rightarrow \omega_v = 1.0$

$h_{wcs}/\ell_w \geq 2.0 \rightarrow \omega_v = 0.9 + n_s/10$ for $n_s \leq 6$

$\omega_v = 1.3 + n_s/30 \leq 1.8$ for $n_s > 6$

where $n_s \geq 0.007 h_{wcs}$

n_s = number of stories above the critical section.



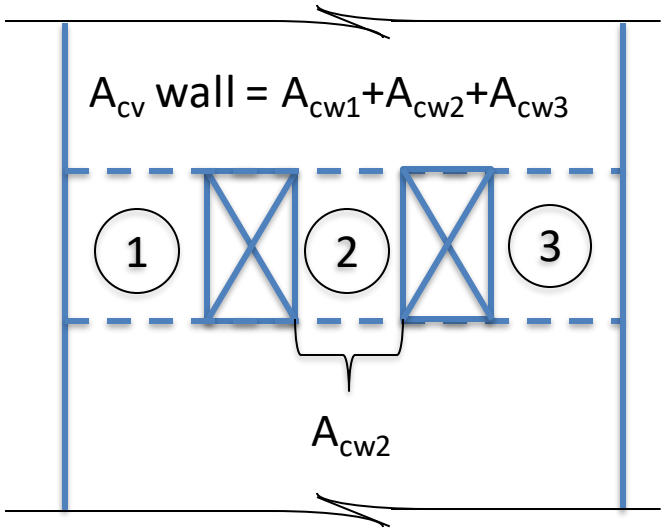
18.10.4.1—Shear strength, V_n

No Change

- The code shows change bars at this location; rewording only
- Shear calculations for Chapters 11 and 18 were harmonized
- 11.5.4.3 is now similar to 18.10.4.1

18.10.4.4—Clarification of A_{cv}

A_{cv} = gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered in the case of walls, and gross area of concrete section in the case of diaphragms. Gross area is total area of the defined section minus area of any openings.

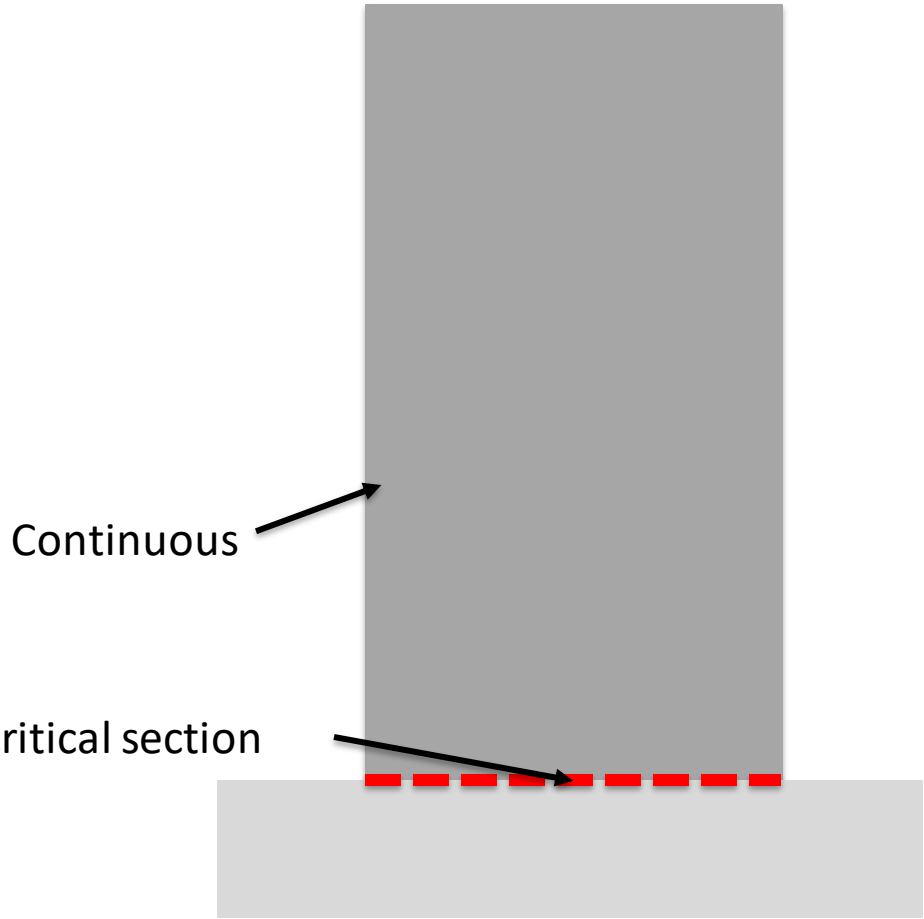


Vertical wall segments

18.10.6.2—Displacement based approach

Boundary elements of special structural walls:

- **Walls or wall piers with $h_{wcs}/\ell_w \geq 2.0$**
- **Continuous**
  - Uniform for full height
- **Single critical (yielding) section**
  - Plastic hinge

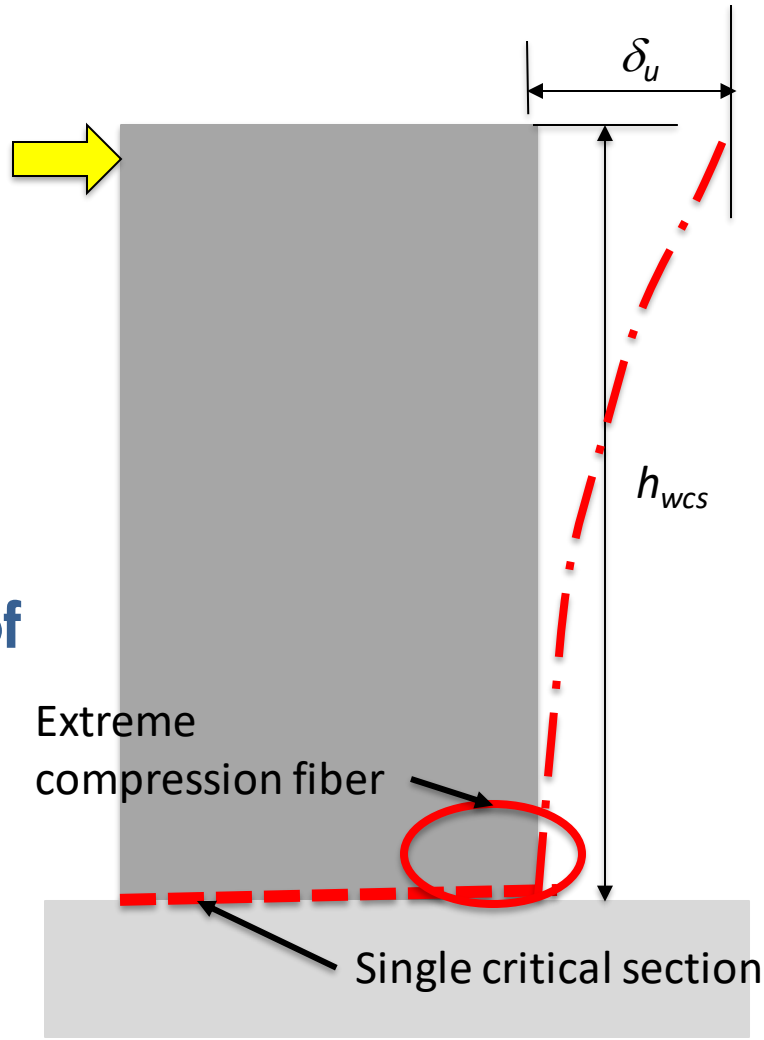


18.10.6.2—Displacement based approach

(a) Compression zone with special boundary elements required if:

$$\frac{1.5\delta_u}{h_{wcs}} \geq \frac{\ell_w}{600c}$$

- **$c = [P_u, \phi M_n]_{max}$ in direction of design displacement δ_u and**
- **$\delta_u / h_{wcs} \geq 0.005$**



18.10.6.2—Displacement based approach

(b) Boundary elements req'd, then (i) and either (ii) or (iii)

i. Transv. reinf. extends above and below critical section $[\ell_w, M_u/4V_u]_{max}$

ii. $b \geq \sqrt{0.025\ell_w c}$

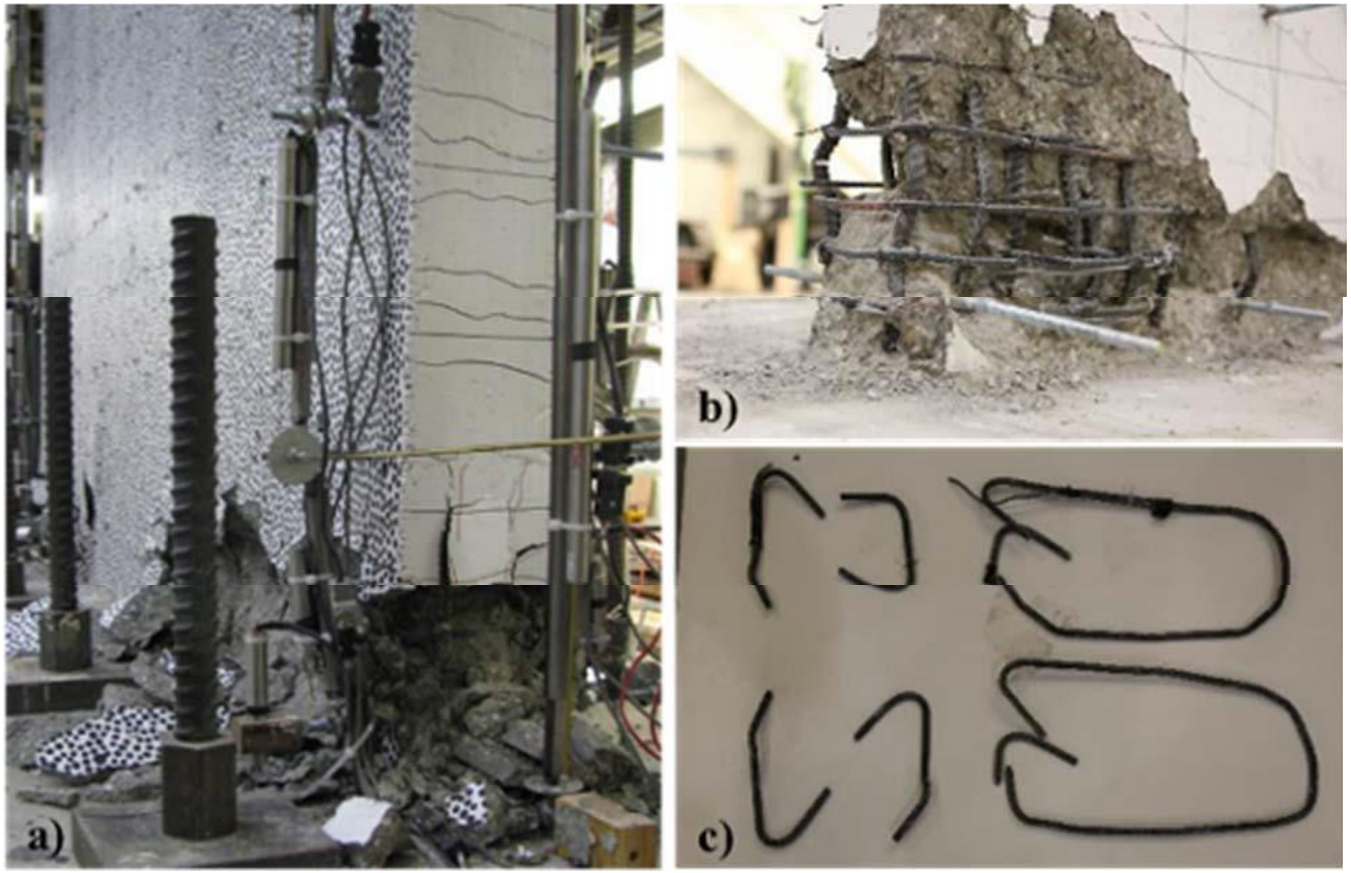
Errata

iii. $\delta_c/h_{wcs} \geq 1.5\,\delta_u / h_{wcs}$, where

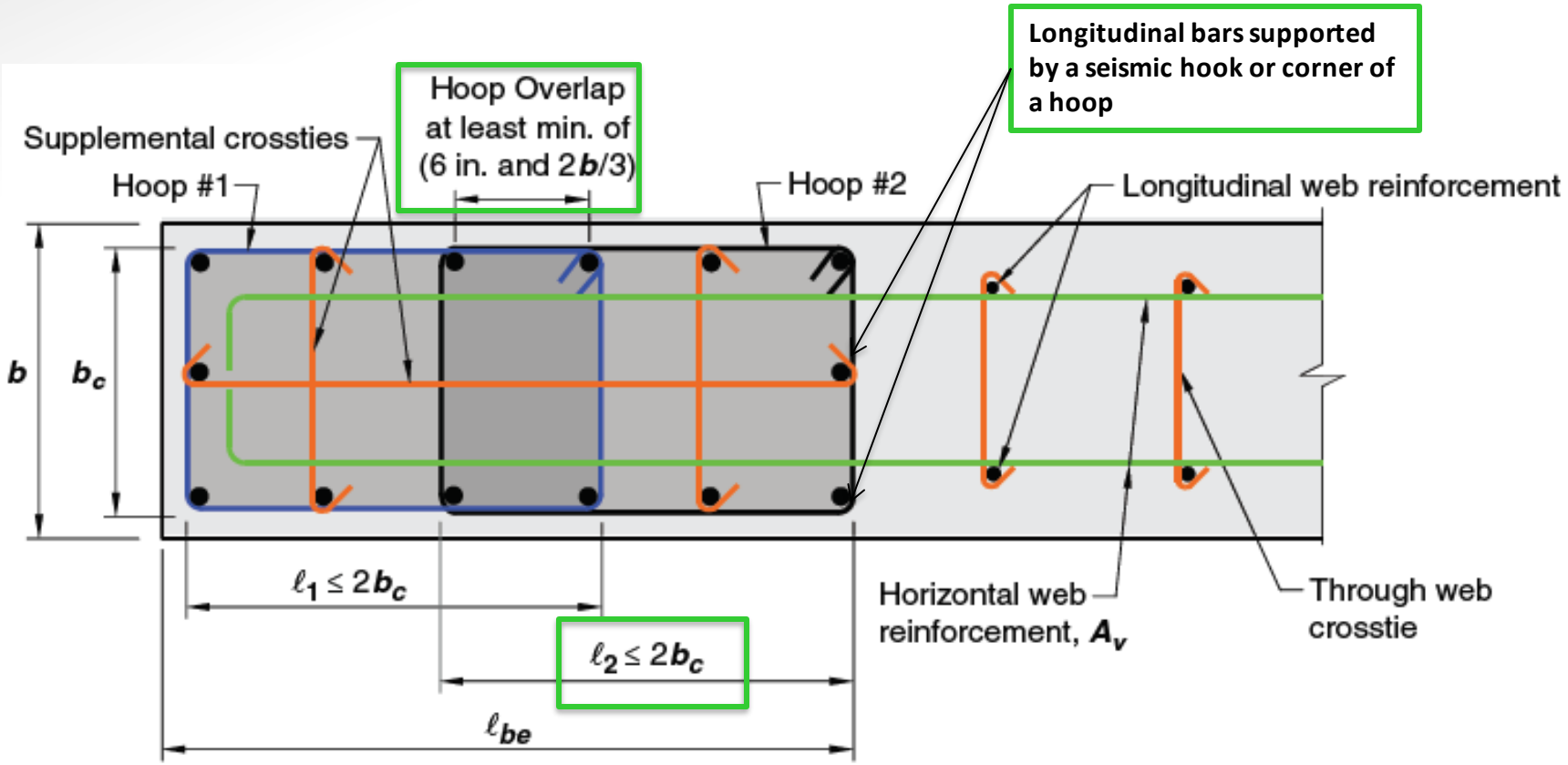
$$\frac{\delta_c}{h_{wcs}} = \frac{1}{100}\left(4 - \frac{1}{50}\left(\frac{\ell_w}{b}\right)\left(\frac{c}{b}\right) - \frac{V_e}{8\sqrt{f_c'}A_{cv}}\right) \geq 0.015$$

18.10.6.4—Special Boundary Elements

- Single perimeter hoops with 90-135 or 135-135 degree crossties, inadequate



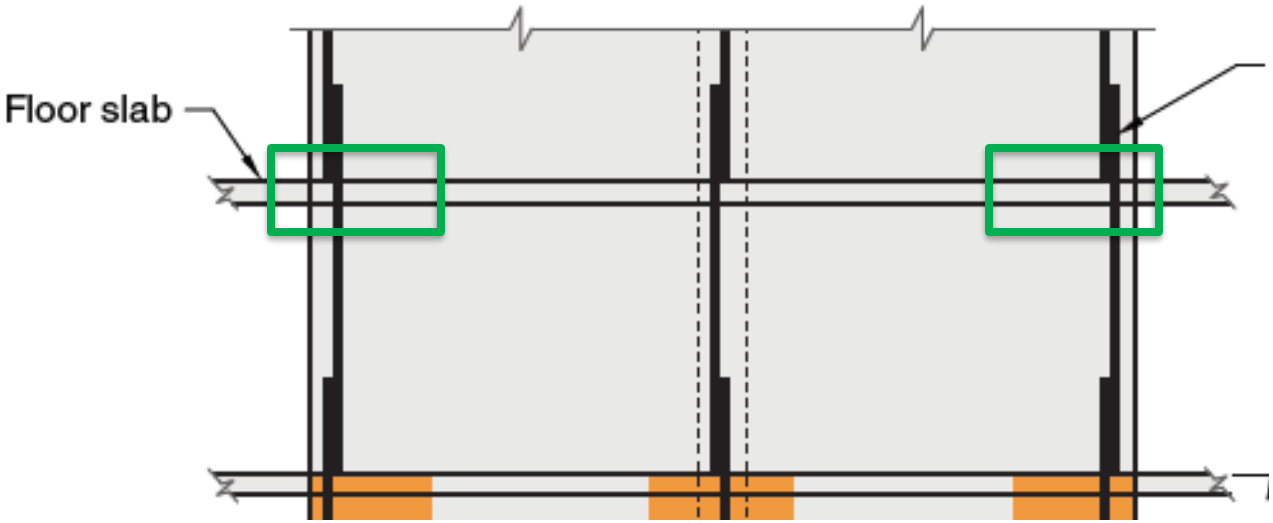
18.10.6.4(f)—Special Boundary Elements



(b) Overlapping hoops with supplemental 135-degree crossties and 135-degree crossties supporting distributed web longitudinal reinforcement

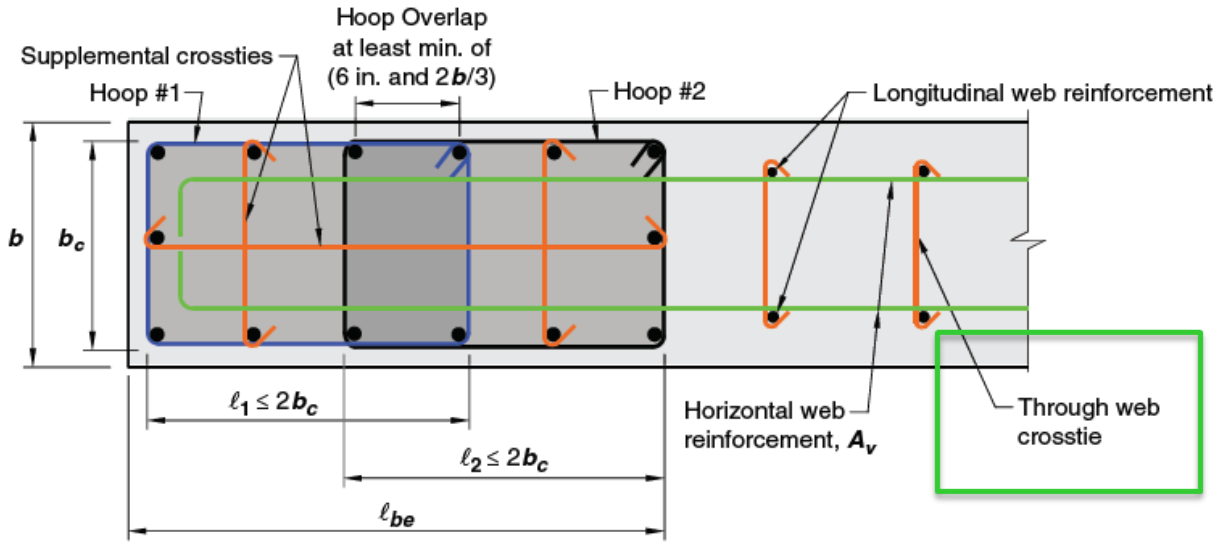
18.10.6.4(h)—Special Boundary Elements

- Concrete within the thickness of the floor system at the special boundary element location shall have specified compressive strength at least 0.7 times f'_c of the wall.



18.10.6.4(i)—Special Boundary Elements

- **18.10.6.4(i) – for a distance specified in 18.10.6.2(b) above and below the critical section, web vertical reinforcement shall have lateral support**
  - crossties vertical spacing, $s_v \leq 12$ in.



(b) Overlapping hoops with supplemental 135-degree crossties and 135-degree crossties supporting distributed web longitudinal reinforcement

18.10.6.5(b)—If the maximum longitudinal ρ at the wall boundary exceeds $400/f_y$

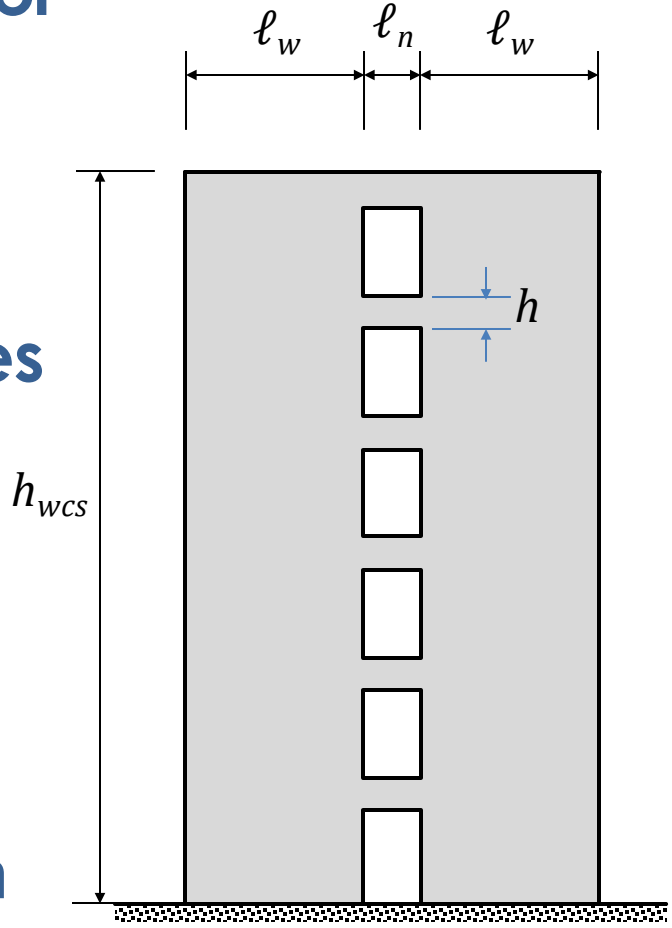
Table 18.10.6.5b—Maximum vertical spacing of transverse reinforcement at wall boundary

Grade of primary flexural reinforcing bar	Transverse reinforcement required	Vertical spacing of transverse reinforcement[1]	
60	Within the greater of ℓ_w and $M_u/4V_u$ above and below critical sections[2]	Lesser of:	6 d_b
			6 in.
	Other locations	Lesser of:	8 d_b
			8 in.
80	Within the greater of ℓ_w and $M_u/4V_u$ above and below critical sections[2]	Lesser of:	5 d_b
			6 in.
	Other locations	Lesser of:	6 d_b
			6 in.
100	Within the greater of ℓ_w and $M_u/4V_u$ above and below critical sections[2]	Lesser of:	4d_b
			6 in.
	Other locations	Lesser of:	6d_b
			6 in.

18.10.9—Ductile Coupled Walls

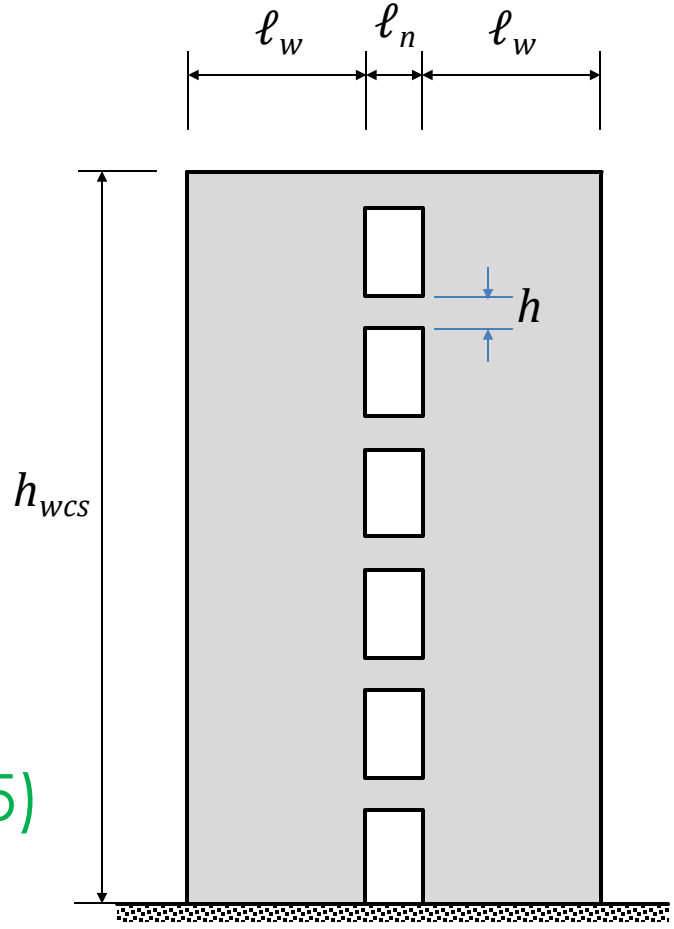
Issues preventing ductile behavior

- Inadequate quantity or distribution of qualifying coupling beams
- Presence of squat walls causes the primary mechanism to be shear and/or strut-and-tie failure in walls
- Coupling beams are inadequately developed to provide full energy dissipation



18.10.9—Ductile Coupled Walls

- **Individual walls satisfy**
  - $h_{wcs}/\ell_w \geq 2$
- **All coupling beams must satisfy:**
  - $\ell_n/h \geq 2$ at all levels
  - $\ell_n/h \leq 5$ at a floor level in at least 90% of the levels of the building
  - Development into adjacent wall segments, $1.25f_y$ (18.10.2.5)



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Foundations

18.13.4—Foundation seismic ties

SDC C through F

- Seismic ties or by other means

SDC D, E, or F, with Site Class E or F

- Seismic ties required

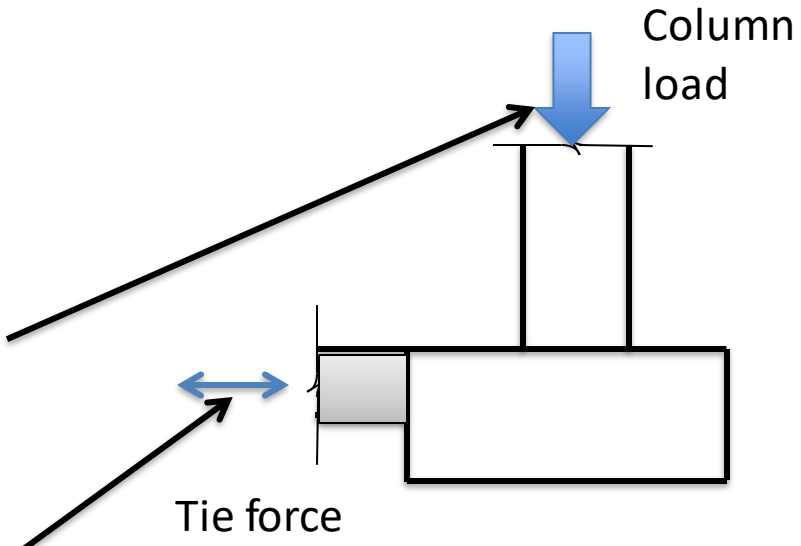
Other means, 18.13.4.3

- Reinforced concrete beams within the slab-on-ground
- Reinforced concrete slabs-on-ground
- Confinement by competent rock, hard cohesive soils, or very dense granular soils
- Other means approved by the building official

18.13.4.3—Seismic ties

Minimum tensile and compressive force in tie

- **Load from pile cap or column**
  - Largest at either end
- **$0.1S_{DS}$ x Column factored dead and factored live load**





18.13.5—Deep foundations

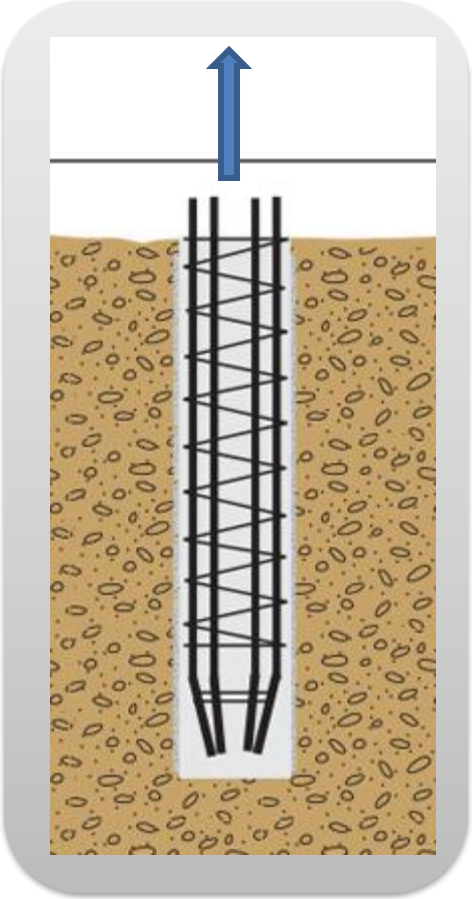
- (a) Uncased CIP concrete drilled or augered piles
- (b) Metal cased concrete piles
- (c) Concrete filled pipe piles
- (d) Precast concrete piles

18.13.5.2—Deep foundations

SDC C through F

- Resisting tension loads

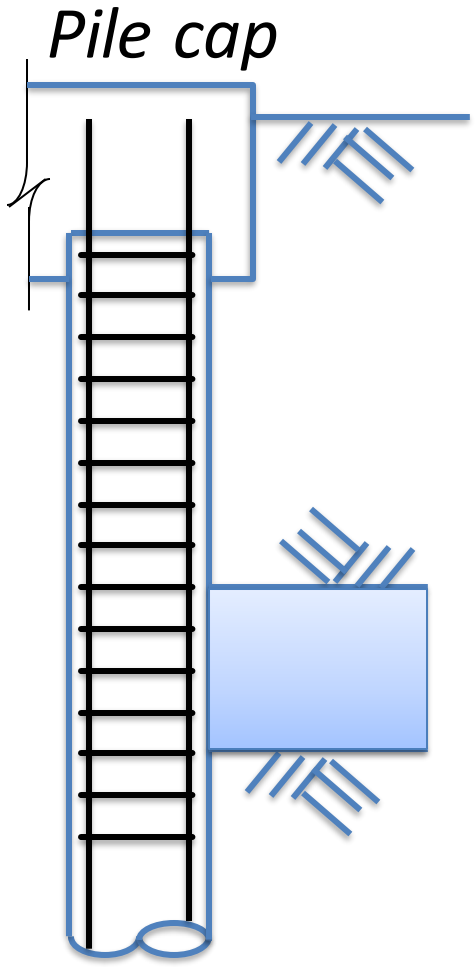
→ Continuous longitudinal reinforcement over full length to resist design tension



18.13.5.3—Deep foundations

SDC C through F

- **Transverse and longitudinal reinforcement to extend:**
  - Over entire unsupported length in air, water, or loose soil not laterally supported



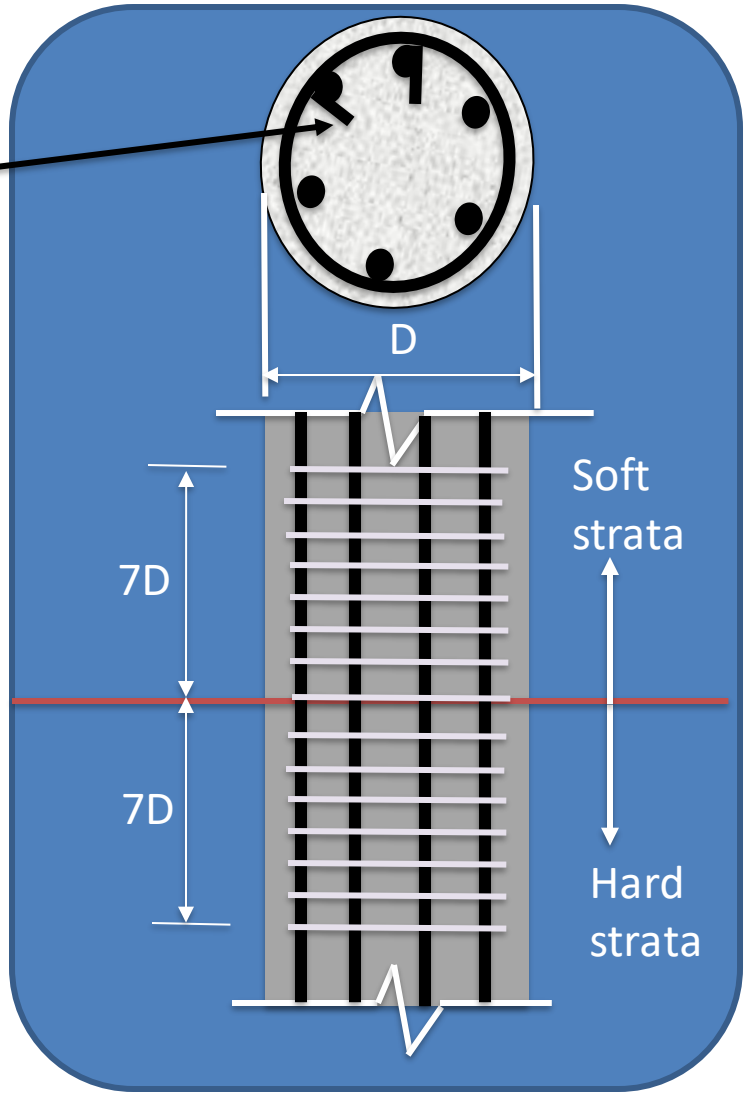
18.13.5.4 and 18.13.5.5—Deep foundations

SDC C through F

- Hoops, spirals or ties terminate in seismic hooks

SDC D, E, or F, with Site Class E or F

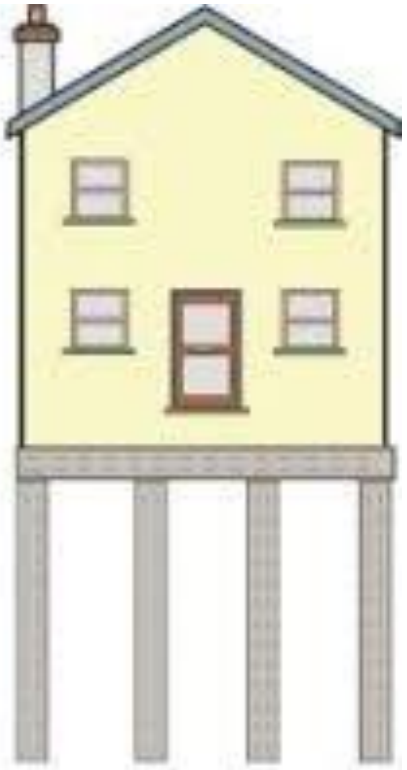
- Transv. reinf. per column req. within seven member diameter
- ASCE 7, soil strata



18.13.5.6—Deep foundations

- **SDC D, E, or F**
  - Piles, piers, or caissons and foundation ties supporting one- and two-story stud bearing walls
  - Exempt from transv. reinf. of 18.13.5.3 through 18.13.5.5

Errata



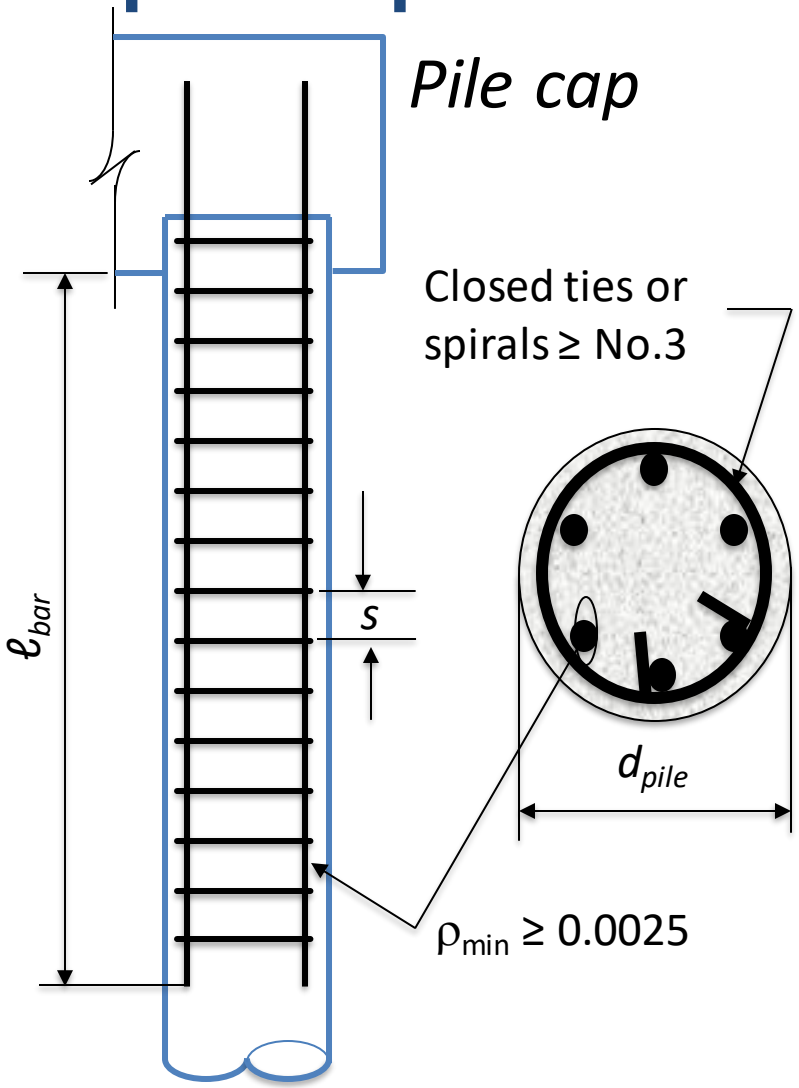
18.13.5.7—Uncased cast-in place piles

SDC C

- $\ell_{bar} \geq$
  - $1/3\ \ell_{pile}$
  - 10 ft
  - $3d_{pile}$
  - Distance to $0.4M_{cr} > M_u$

- **Transverse confinement zone**
  - 3 d_{pile} from bottom of pile cap
  - $s \leq 6$ in.; $8d_b$ long. bar

- **Extended trans. reinf.**
  - $s \leq 16d_b$ long. bar



ℓ_{bar} = minimum reinforced pile length

18.13.5.7—Uncased cast-in place piles

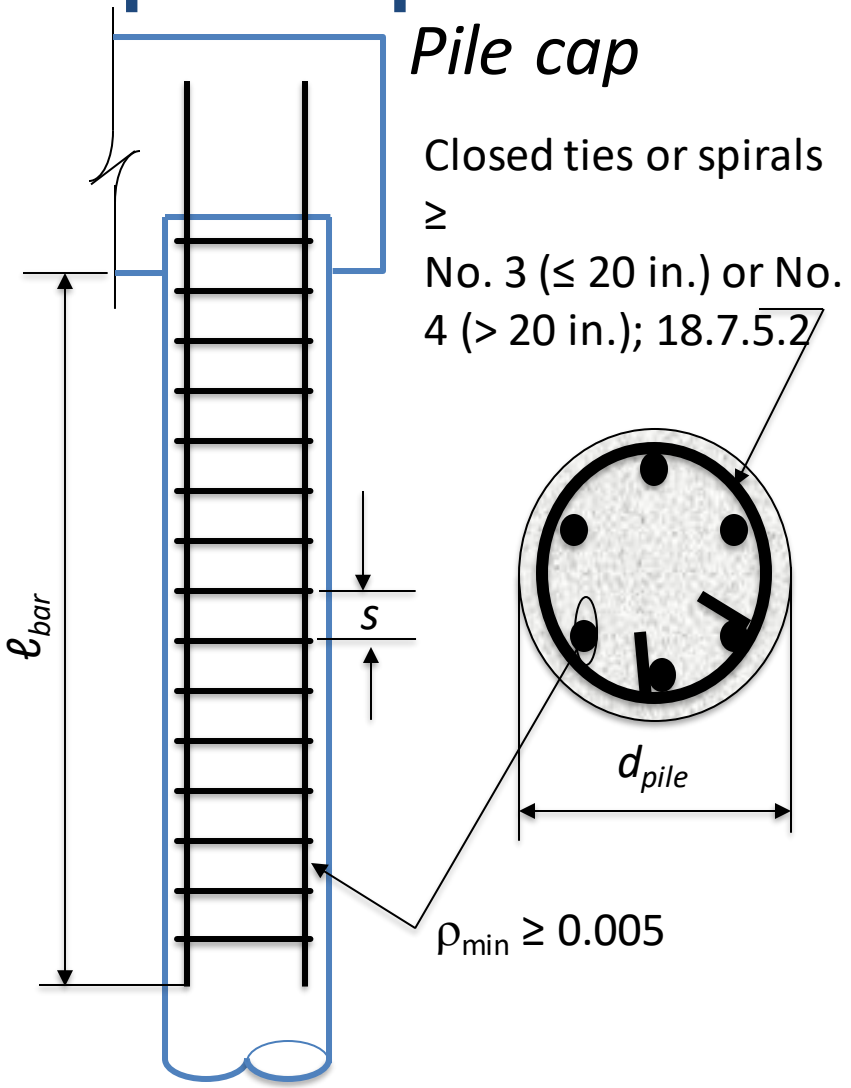
SDC D, E, and F with Site Class A, B, C, and D

- $\ell_{bar} \geq$
  - $1/2\ \ell_{pile}$
  - 10 ft
  - $3d_{pile}$
  - Distance to $0.4M_{cr} > M_u$

- **Transverse confinement zone**
  - 3 d_{pile} from bottom of pile cap
  - *s of* 18.7.5.3
  - $\rho_{min} \geq 0.06\ f_c'/f_{yt}$

- **Extended trans. reinf.**

$s \leq$
- $12d_b$ long. bar
- $0.5d_{pile}$
- 12 in.



ℓ_{bar} = minimum reinforced pile length

18.13.5.7—Uncased cast-in place piles

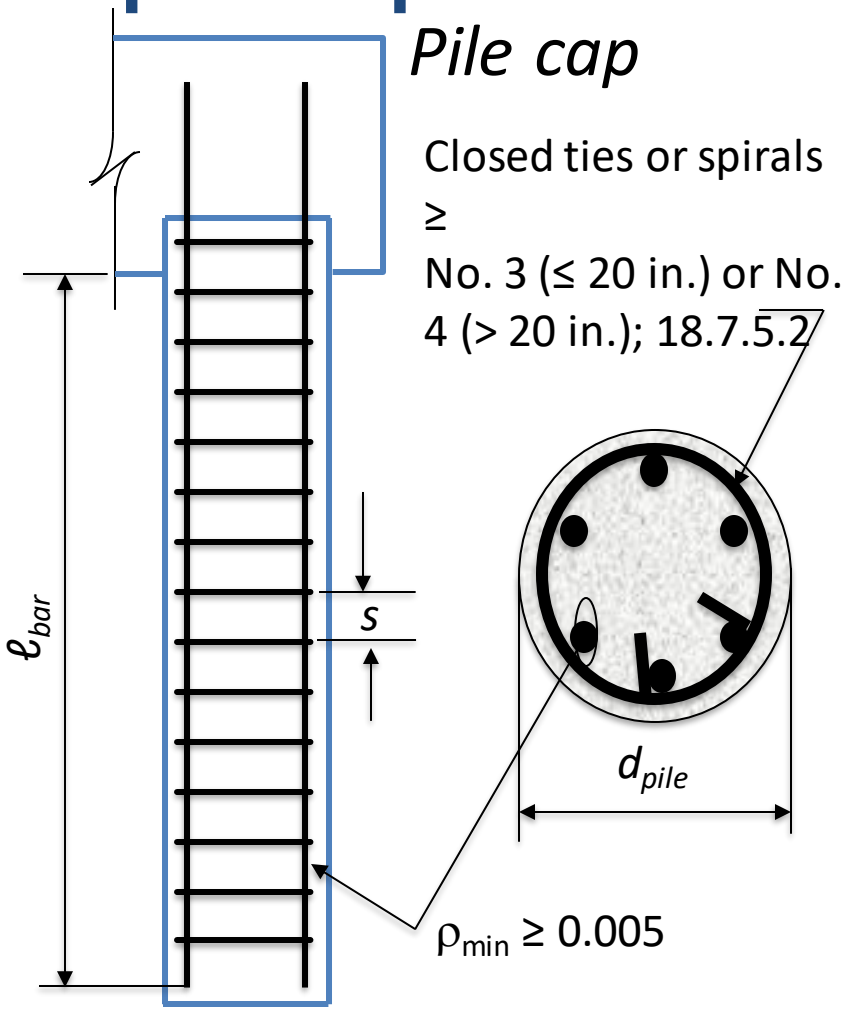
SDC D, E, and F with Site Class E and F

- ℓ_{bar} Full length of pile (some exceptions)

- Transverse confinement zone
  - 7 d_{pile} from bottom of pile cap
  - s of 18.7.5.3
  - $\rho_{min} \geq 0.06\, f_c'/f_{yt}$

- Extended trans. reinf.

$$s \leq \begin{cases} 12 d_b \text{ long. bar} \\ 0.5 d_{pile} \\ 12 \text{ in.} \end{cases}$$

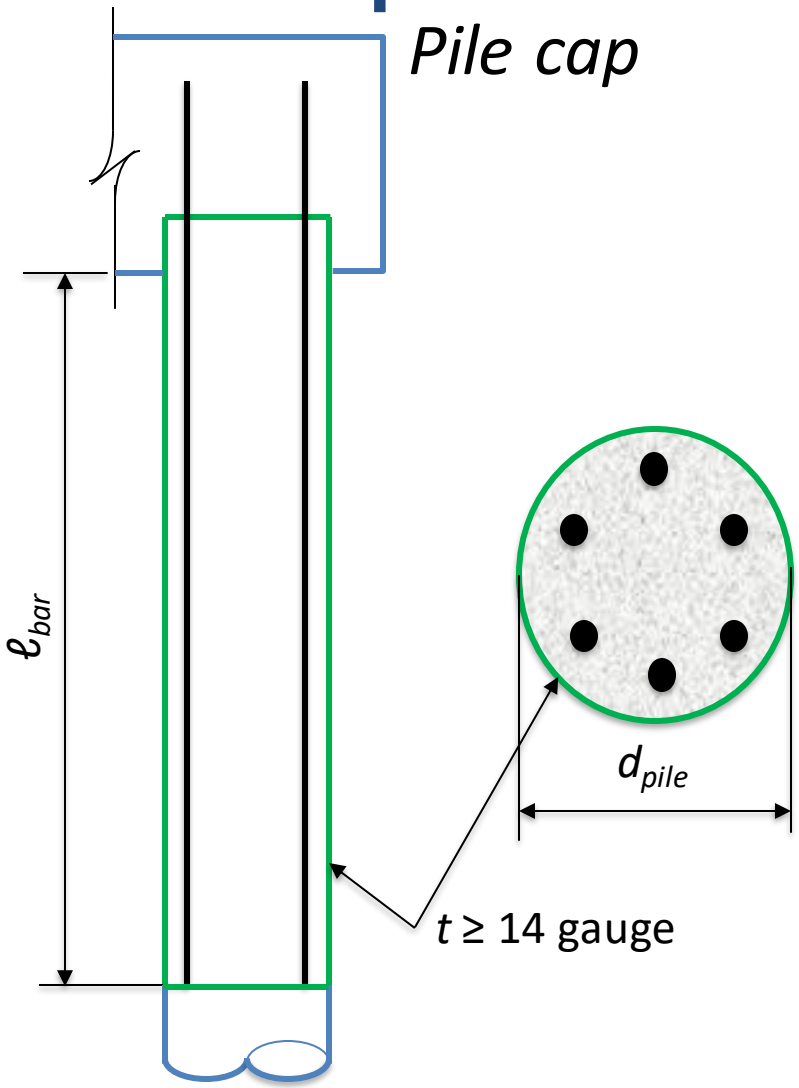


ℓ_{bar} = minimum reinforced pile length

18.13.5.8—Metal cased concrete piles

SDC C through F

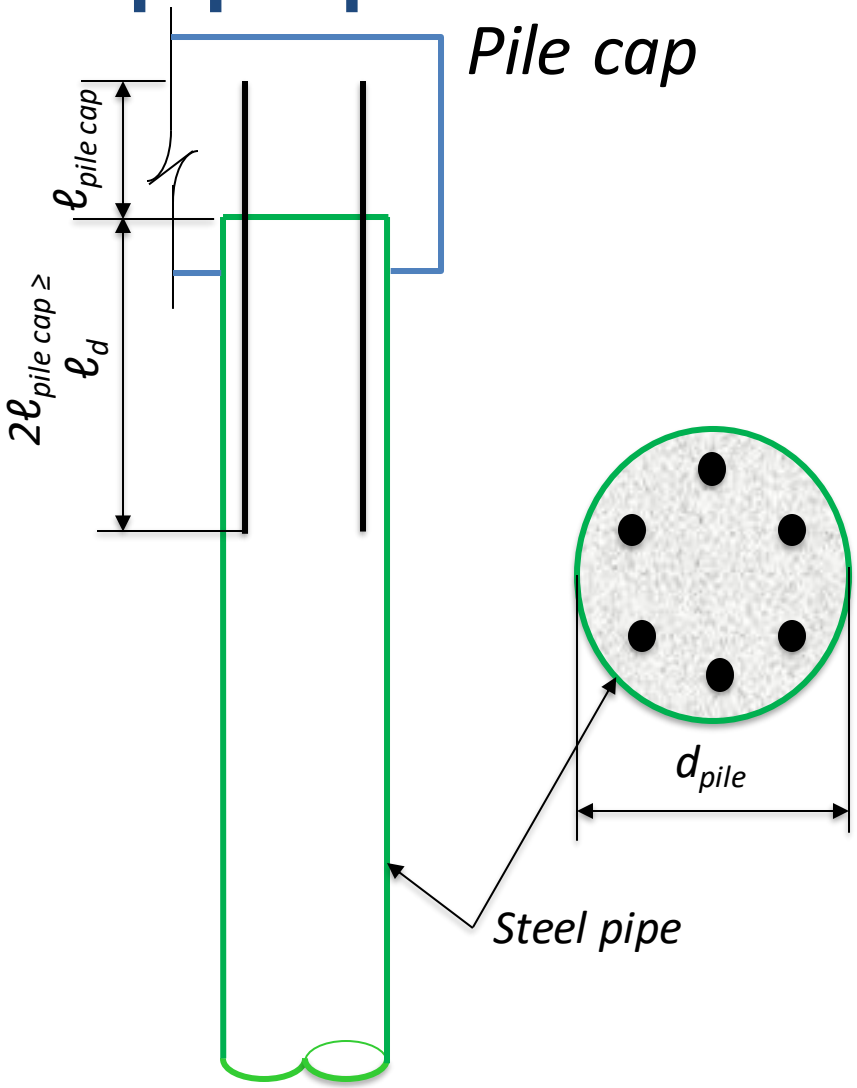
- Longitudinal same as uncased piles
- Metal casing replaces transverse reinforcement in uncased piles
- Extend casing for ℓ_{bar}



18.13.5.9—Concrete-filled pipe piles

SDC C through F

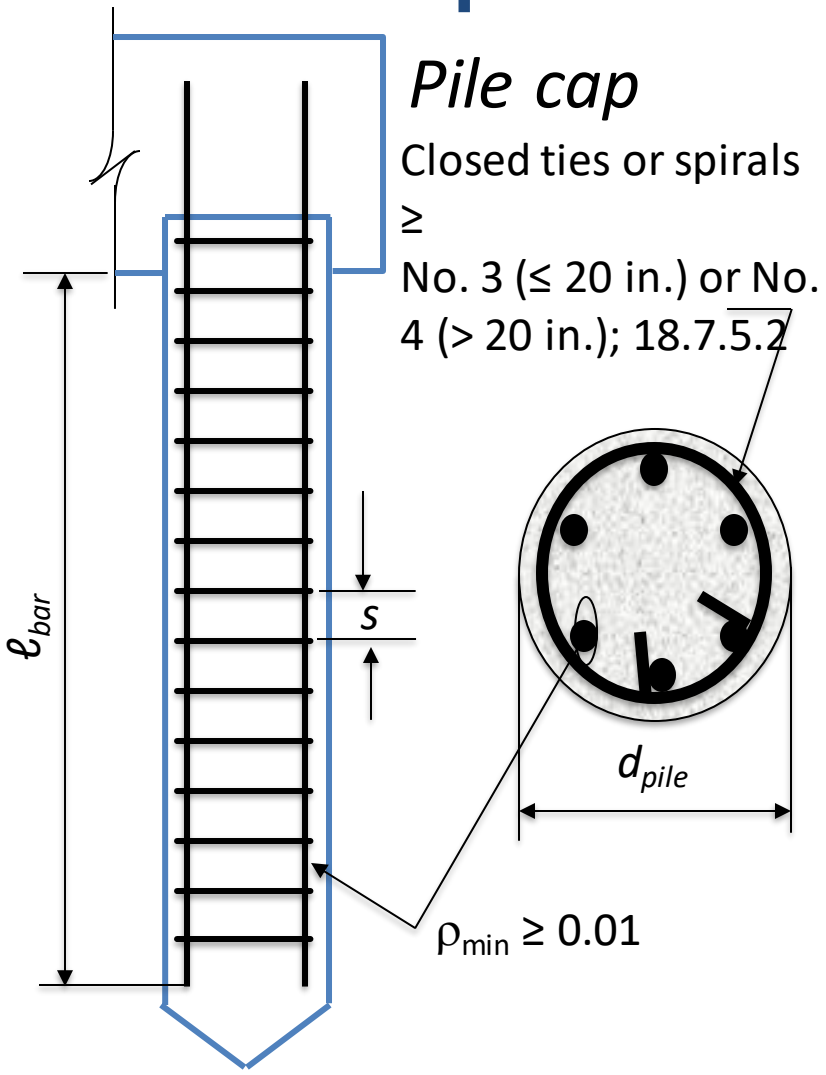
- $\rho_{min} \geq 0.01$
- $\ell_{d,pile} \geq \begin{cases} 2\ell_{pilecap} \\ \ell_{dt,bar} \end{cases}$



18.13.5.10—Precast nonprestressed piles

SDC C

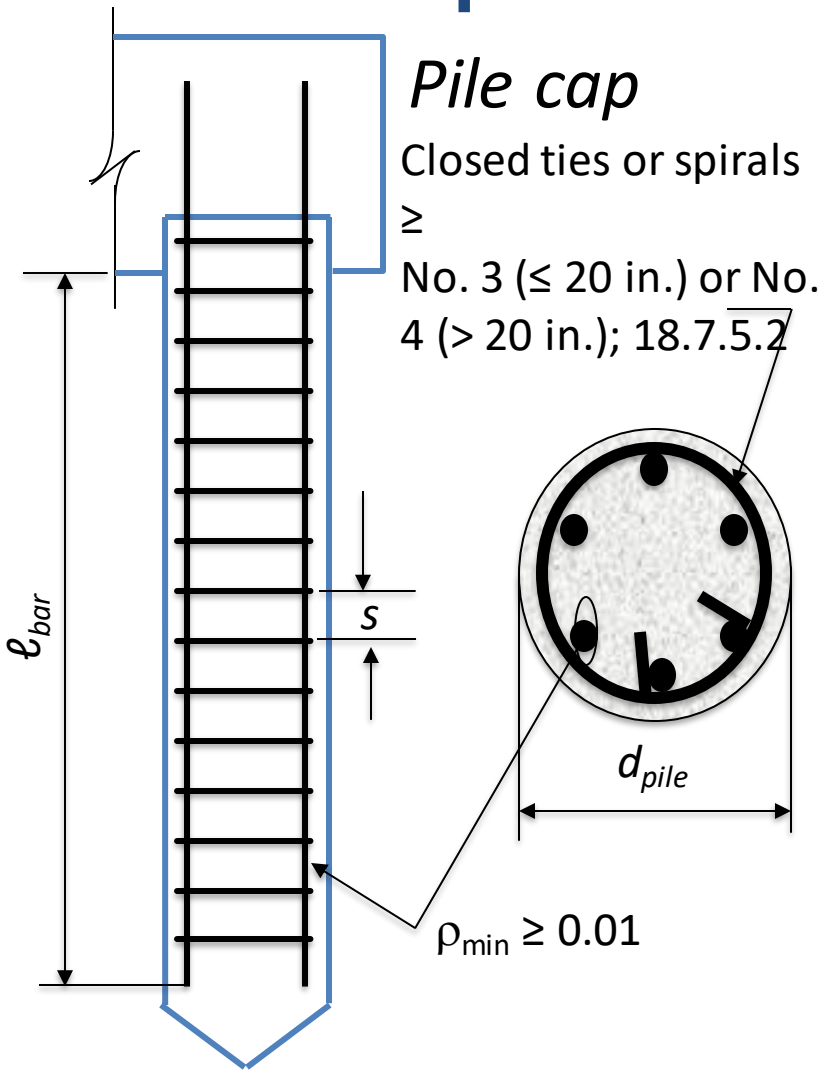
- ℓ_{bar} Full length of pile
- Transverse confinement zone
  - 3 d_{pile} from bottom of pile cap
  - $s \leq 6$ in.; $8d_b$ long. bar
- Extended trans. reinf.
  - $s \leq 6$ in.



18.13.5.10—Precast nonprestressed piles

SDC D, E, and F

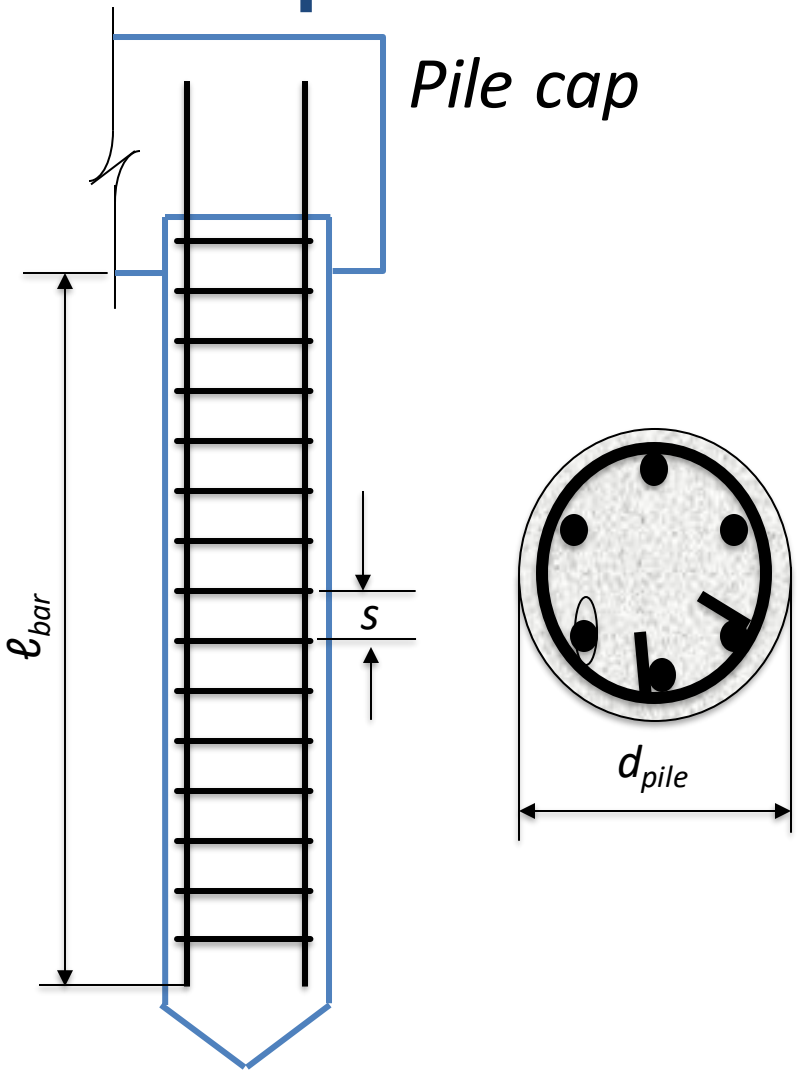
- Same as SDC C
- Satisfy Table 18.13.5.7.1 for SDC D, E, and F



18.13.5.10—Precast prestressed piles

SDC C through F

- Satisfy 18.13.5.10.4 through 18.13.5.10.6
- Minimum amount and spacing of transverse reinforcement



18.13.6—Anchorage of piles, piers and caissons

SDC C—F

- Tension loads: load path to piles, piers, or caissons
- Transfer to longitudinal reinforcement in deep foundation

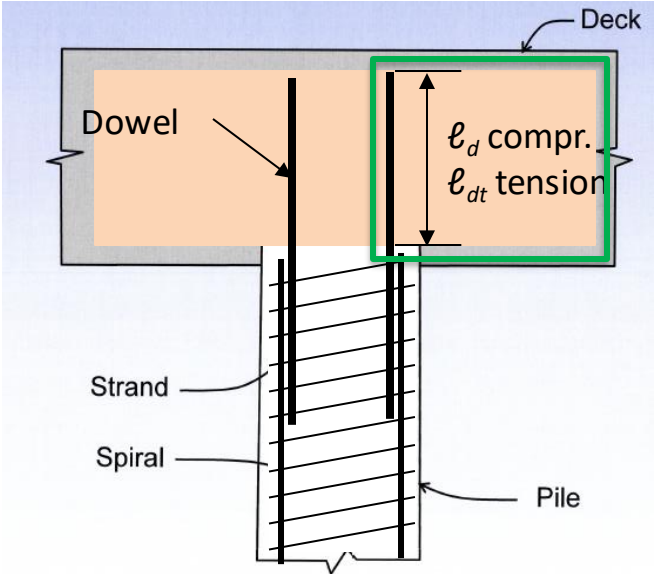

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18.13.6—Anchorage of piles, piers and caissons

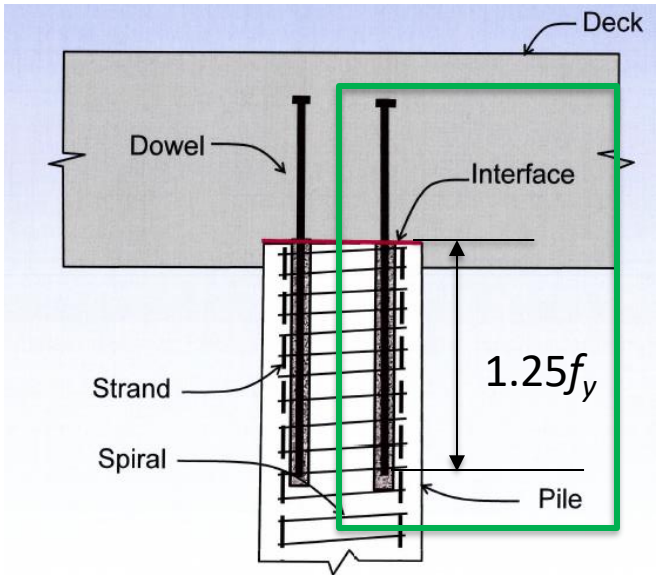
18.13.6.2 SDC C—F

- Anchor dowel between piles and pile cap



18.13.6.3 SDC D—F

- If tension forces and dowel post-installed in precast pile
- Grouting system to develop min. 1.25 f_y (shown by test)



21.2.4.3—ϕ, Foundation elements

SDC C—F

- **For foundation elements supporting the primary seismic-force-resisting system**
- **ϕ for shear shall ≤ the least value of**
  - ϕ for shear used for special column
  - ϕ for shear used for special wall

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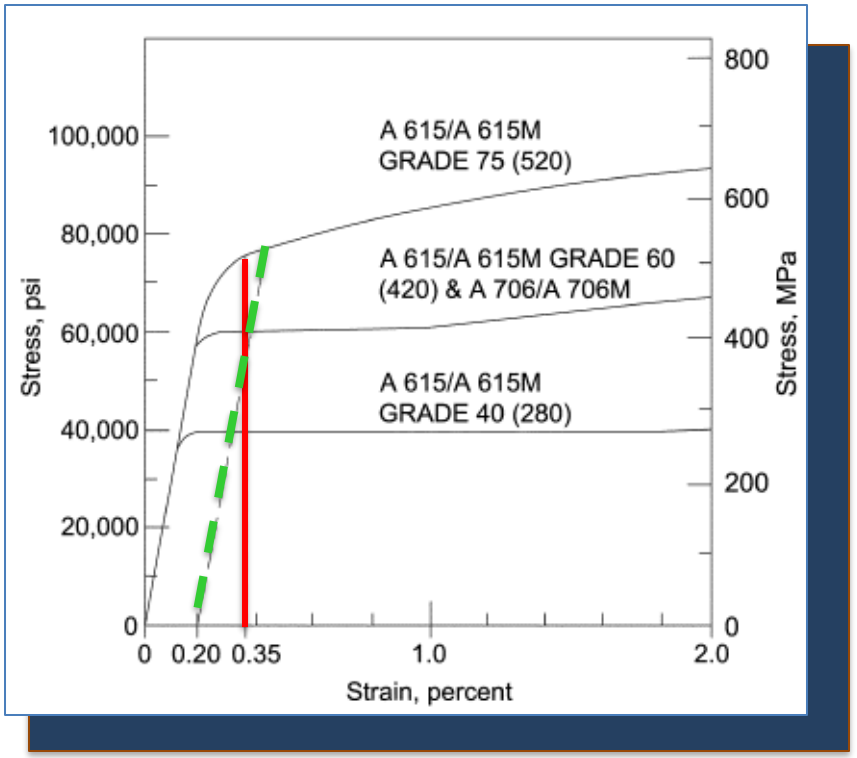
Changes to the Concrete Design Standard



High-Strength Reinforcement

Ch. 20 – Yield strength determination

- **318-19, 20.2.1.2: Nonprestressed bar yield strength determination:**
  - The yield point by the halt-of-force method
  - The offset method, using 0.2 percent offset
- **20.2.1.3**
  - A615 and A706 additional requirements



Ch. 3 – Update of ASTM A615-18$^{\varepsilon 1}$

- **Latest ASTM A615 allows:**
  - Gr. 100
  - Bars up to No. 20
- **ACI 318-19 allows**
  - No. 18 and smaller
  - Gr. 80 & 100 with restrictions
- **No. 20 not acceptable:**
  - Development length
  - Bar bends



Table 20.2.2.4(a)

- **Main chan**
  - Gr. 80
  - Gr. 100
  - Footnotes
  - Clarifications

Table 20.2.2.4(a)—Nonprestressed deformed reinforcement

Usage	Application		Maximum value of f_y or f_{yt} permitted for design calculations, psi	Applicable ASTM specification			
				Deformed bars	Deformed wires	Welded wire reinforcement	Welded deformed bar mats
Flexure; axial force; and shrinkage and temperature	Special seismic systems	Special moment frames	80,000	A706[2]	Not permitted	Not permitted	Not permitted
		Special structural walls[1]	100,000				
	Other		100,000[3] [4]	A615, A706, A955, A996, A1035	A1064, A1022	A1064, A1022	A184[5]
Lateral support of longitudinal bars; or concrete confinement	Special seismic systems		100,000	A615, A706, A955, A996, A1035	A1064, A1022	A1064[6], A1022[6]	Not permitted
	Spirals		100,000	A615, A706, A955, A996, A1035	A1064, A1022	Not permitted	Not permitted
	Other		80,000	A615, A706, A955, A996	A1064, A1022	A1064, A1022	Not permitted
Shear	Special seismic systems[7]	Special moment frames[8]	80,000	A615, A706, A955, A996	A1064, A1022	A1064[6], A1022[6]	Not permitted
		Special structural walls[9]	100,000				
	Spirals		60,000	A615, A706, A955, A996	A1064, A1022	Not permitted	Not permitted
	Shear friction		60,000	A615, A706, A955, A996	A1064, A1022	A1064, A1022	Not permitted
	Stirrups, ties, hoops		60,000	A615, A706, A955, A996, A1035	A1064, A1022	A1064 and A1022 welded plain wire	Not permitted
			80,000	Not permitted	Not permitted	A1064 and A1022 welded deformed wire	Not permitted
Torsion	Longitudinal and transverse		60,000	A615, A706, A955, A996	A1064, A1022	A1064, A1022	Not permitted
Anchor reinforcement	Special seismic systems		80,000	A706[2]	Not permitted	Not permitted	Not permitted
	Other		80,000	A615, A706, A955, A996	A1064, A1022	A1064, A1022	A184[5]
Regions designed using strut-and-tie method	Longitudinal ties		80,000	A615, A706, A955, A996	A1064, A1022	A1064, A1022	Not permitted
	Other		60,000				

[1]All components of special structural walls, including coupling beams and wall piers.

[2]ASTM A615 Grade 60 shall be permitted if requirements of 20.2.2.5(b) are satisfied.

[3]In slabs and beams not part of a special seismic system, bars that pass through or extend from special structural walls shall satisfy 20.2.2.5.

[4]Longitudinal reinforcement with $f_y > 80{,}000$ psi is not permitted for intermediate moment frames and ordinary moment frames resisting earthquake demands E.

[5]Welded deformed bar mats shall be permitted to be assembled using only ASTM A615 or A706 deformed bars of Grade 60 or Grade 80.

[6]ASTM A1064 and A1022 are not permitted in special seismic systems if the weld is required to resist stresses in response to confinement, lateral support of longitudinal bars, shear, or other actions.

[7]This application also includes shear reinforcement with a maximum value of 80,000 psi f_y or f_{yt} permitted for design calculations for diaphragms and foundations for load combinations including earthquake forces if part of a building with a special seismic system.

[8]Shear reinforcement in this application includes stirrups, ties, hoops, and spirals in special moment frames.

[9]Shear reinforcement in this application includes all transverse reinforcement in special structural walls, coupling beams, and wall piers. Diagonal bars in coupling beams shall comply with ASTM A706 or Footnote [2].

Ch. 20 – Steel Reinforcement Properties

Flexure; axial force; and shrinkage and temperature	Special seismic systems	Special moment frames	80,000	A706[2]
		Special structural walls[1]	100,000	
	Other		100,000[3] [4]	A615, A706, A955, A996, A1035

[1]All components of special structural walls, including coupling beams and wall piers.

[2]ASTM A615 Grade 60 shall be permitted if requirements of 20.2.2.5(b) are satisfied.

[3]In slabs and beams not part of a special seismic system, bars that pass through or extend from special structural walls shall satisfy 20.2.2.5.

[4]Longitudinal reinforcement with f_y > 80,000 psi is not permitted for intermediate moment frames and ordinary moment frames resisting earthquake demands *E*.



Ch. 20 –Seismic Requirements for A615 Gr. 60

- **Section 20.2.2.5 specifies**
  - ASTM A706 Gr. 60 allowed
  - Requirements for ASTM A615, Gr. 60
- **Section 20.2.2.5(a) permits ASTM A706**
  - Grade 60
  - Grade 80
  - Grade 100
  - (as discussed previously)

Ch. 20 –Seismic Requirements for A615 Gr. 60

- **Section 20.2.2.5(b) permits ASTM A615 Grade 60 if:**
  - f_y,actual ≤ f_y + 18,000 psi
  - Provides adequate ductility (min. $f_t/f_y \geq 1.25$)
  - Min. fracture elongation in 8 in. (10-14%)
  - Minimum uniform elongation (6-9%)
- **Section 20.2.2.5(b) provides the A706 elongation properties**

Ch. 20 – Seismic Requirements for A615

- For seismic design ASTM A615 GR. 80 and 100 are not permitted



Ch. 20 – Steel Reinforcement Properties

Shear	Special seismic systems[7]	Special moment frames[8]	80,000	A615, A706, A955, A996
		Special structural walls[9]	100,000	
	Spirals		60,000	A615, A706, A955, A996
	Shear friction		60,000	A615, A706, A955, A996
	Stirrups, ties, hoops		60,000	A615, A706, A955, A996, A1035
			80,000	Not permitted
Torsion	Longitudinal and transverse		60,000	A615, A706, A955, A996

[7]This application also includes shear reinforcement with a maximum value of 80,000 psi f_y or f_{yt} permitted for design calculations for diaphragms and foundations for load combinations including earthquake forces if part of a building with a special seismic system.

[8]Shear reinforcement in this application includes stirrups, ties, hoops, and spirals in special moment frames.

[9]Shear reinforcement in this application includes all transverse reinforcement in special structural walls, coupling beams, and wall piers. Diagonal bars in coupling beams shall comply with ASTM A706 or Footnote [2].

Ch. 20 – Steel Reinforcement Properties

Anchor reinforcement	Special seismic systems	80,000	A706[2]
	Other	80,000	A615, A706, A955, A996
Regions designed using strut-and-tie method	Longitudinal ties	80,000	A615, A706, A955, A996
	Other	60,000	

[1]All components of special structural walls, including coupling beams and wall piers.

[2]ASTM A615 Grade 60 shall be permitted if requirements of 20.2.2.5(b) are satisfied.

[3]In slabs and beams not part of a special seismic system, bars that pass through or extend from special structural walls shall satisfy 20.2.2.5.

[4]Longitudinal reinforcement with $f_y > 80{,}000$ psi is not permitted for intermediate moment frames and ordinary moment frames resisting earthquake demands E.

Ch. 26 – Tolerances for seismic hoops 26.6.2.1(c)

(c) Tolerance for spacing of hoops in members of intermediate and special seismic systems:

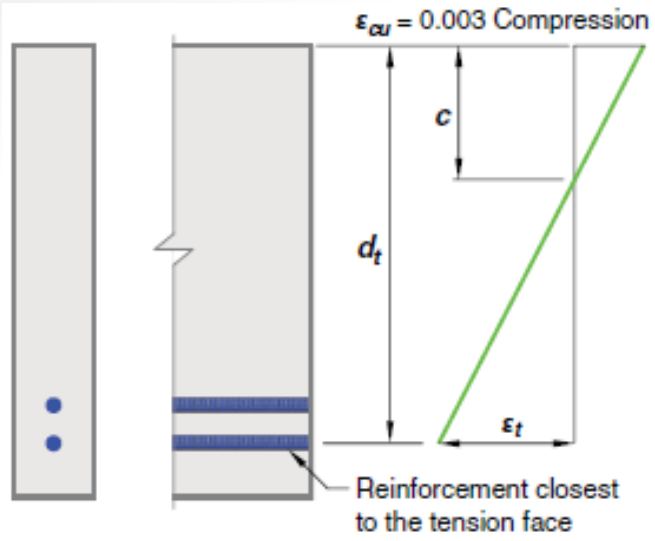
(1) Lesser of +1-1/2 in. and +1.5d_b of the smallest longitudinal bar.

(2) Lesser of –1 in. per ft of least side dimension of member and –3 in.

(3) Spacing adjustments shall result in no more than two hoops being in contact with each other.

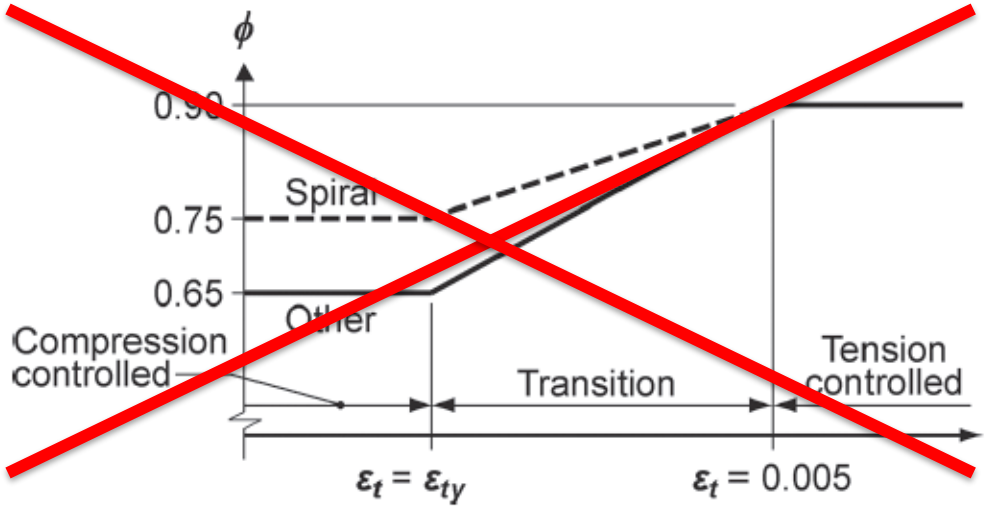
Design limits

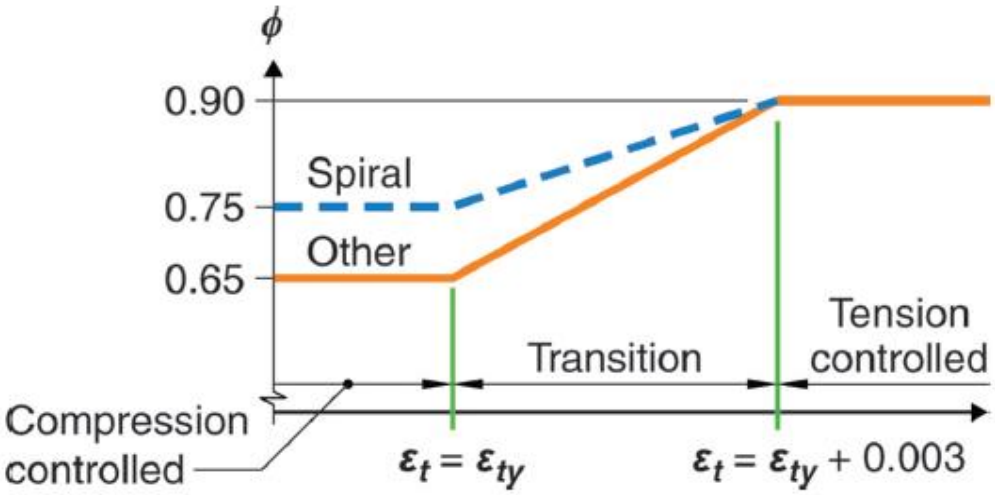
ACI 318-14 ACI 318-19



~~$\varepsilon_t \geq 0.005$~~

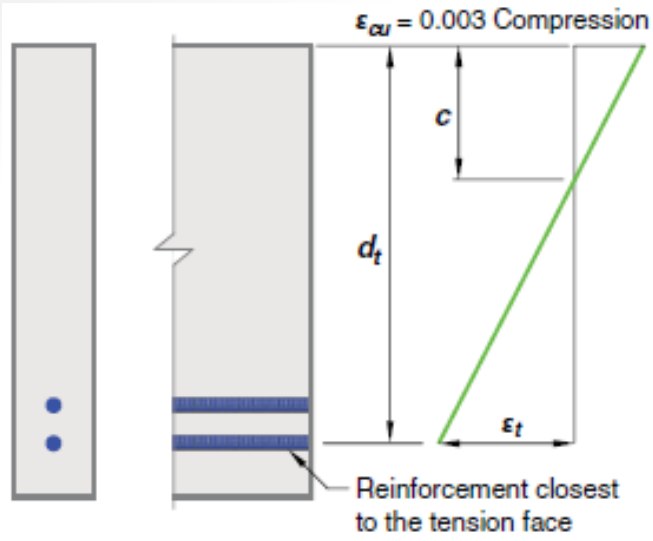
$\varepsilon_t \geq (\varepsilon_{ty} + 0.003)$



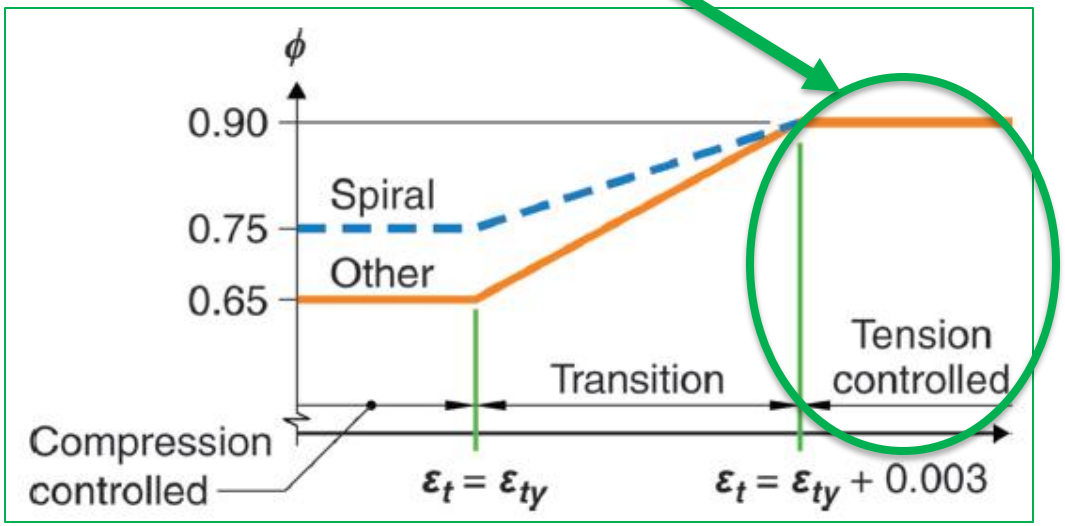


Design limits

ACI 318-19



ACI 318-19 Provisions 7.3.3.1, 8.3.3.1, and 9.3.3.1 require slabs and beams be tension controlled



$$\varepsilon_t \geq (\varepsilon_{ty} + 0.003)$$

$$\varepsilon_{ty} = \frac{f_y}{E_s}$$

Design limits

ACI 318-14

Table 8.4.2.3.4—Maximum modified values of γ_f for nonprestressed two-way slabs

Column location	Span direction	v_{ug}	ε_t (within b_{slab})	Maximum modified γ_f
Corner column	Either direction	$\leq 0.5\phi v_c$	≥ 0.004	1.0
Edge column	Perpen-dicular to the edge	$\leq 0.75\phi v_c$	≥ 0.004	1.0
	Parallel to the edge	$\leq 0.4\phi v_c$	≥ 0.010	$\dfrac{1.25}{1+\left(\frac{2}{3}\right)\sqrt{\frac{b_1}{b_2}}} \leq 1.0$
Interior column	Either direction	$\leq 0.4\phi v_c$	≥ 0.010	$\dfrac{1.25}{1+\left(\frac{2}{3}\right)\sqrt{\frac{b_1}{b_2}}} \leq 1.0$

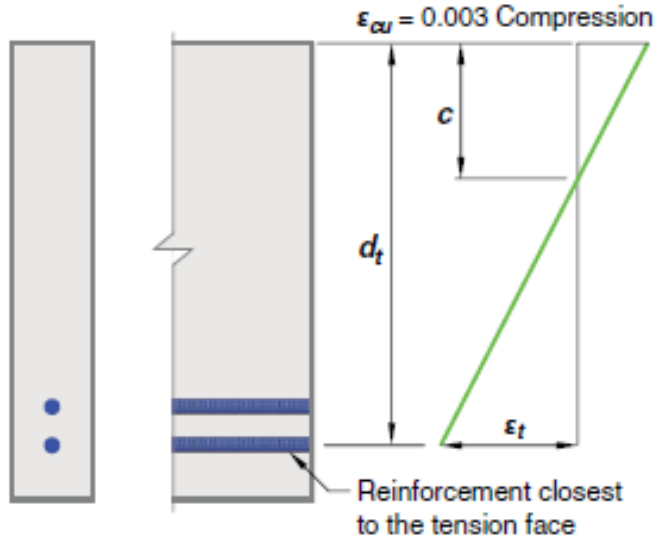
Design limits

ACI 318-19

Table 8.4.2.2.4—Maximum modified values of γ_f for nonprestressed two-way slabs

Column location	Span direction	v_{uv}	ε_t (within b_{slab})	Maximum modified γ_f
Corner column	Either direction	$\leq 0.5\phi v_c$	$\geq \varepsilon_{ty} + 0.003$	1.0
Edge column	Perpendicular to the edge	$\leq 0.75\phi v_c$	$\geq \varepsilon_{ty} + 0.003$	1.0
	Parallel to the edge	$\leq 0.4\phi v_c$	$\geq \varepsilon_{ty} + 0.008$	$\dfrac{1.25}{1+\left(\frac{2}{3}\right)\sqrt{\frac{b_1}{b_2}}} \leq 1.0$
Interior column	Either direction	$\leq 0.4\phi v_c$	$\geq \varepsilon_{ty} + 0.008$	$\dfrac{1.25}{1+\left(\frac{2}{3}\right)\sqrt{\frac{b_1}{b_2}}} \leq 1.0$

Design limits



$$\varepsilon_{ty} = \frac{f_y}{E_s}$$

Reinforcement ratio, ρ_{tcl}

	f'_c = 4000 psi	f'_c = 10,000 psi
GR 60 $\varepsilon_t \geq 0.0051$	1.79%	3.42%
GR 80 $\varepsilon_t \geq 0.00575$	1.24%	2.37%
GR 100 $\varepsilon_t \geq 0.0065$	0.92%	1.75%

Design limits



16 x 24 in. beam

d = 21 in.

f'_c = 4000 psi

GR 60

$A_{s,tcl}$ = 6 in.^{2}

$M_{n,tcl}$ = 544 ft-kip

GR 100

$A_{s,tcl}$ = 3.1 in.^{2}

$M_{n,tcl}$ = 479 ft-kip

Approximately 50% of reinforcement achieved 88% of nominal moment

Reinforcement ratio, ρ_{tcl}

Grade	f'_c = 4 ksi	f'_c = 10 ksi
60	1.79%	3.42%
80	1.24%	2.37%
100	0.92%	1.75%

ACI 318-19

American Concrete Institute
Always advancing

Changes to the Concrete Design Standard



Development Length

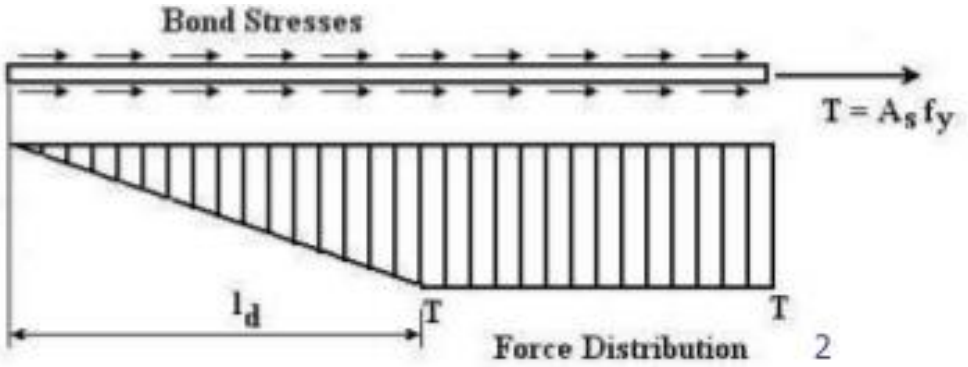


Development Length

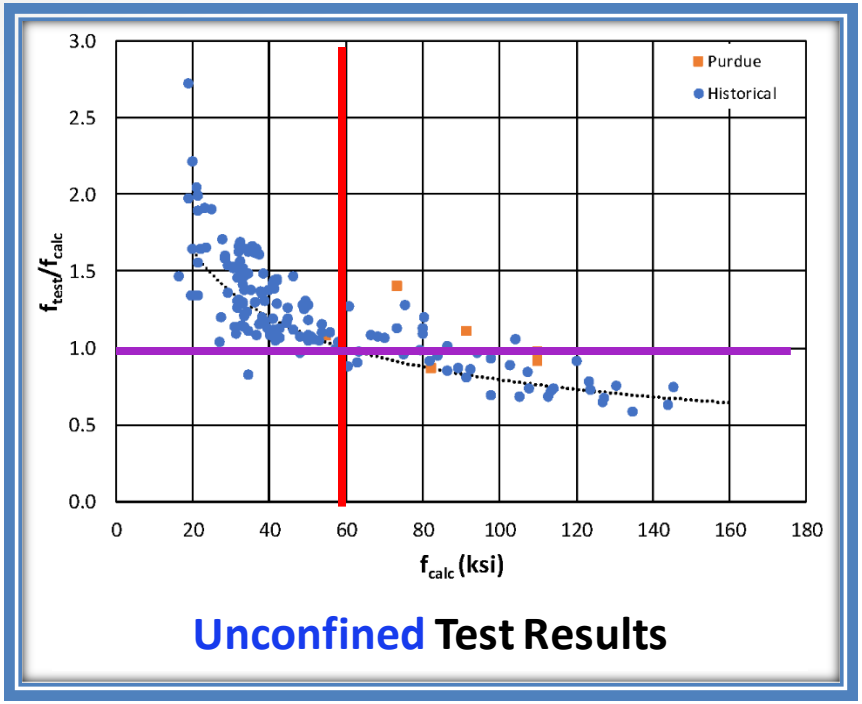
- **Deformed Bars and Deformed Wires in Tension**
  - Simple modification to 318-14
  - Accounts for Grade 80 and 100
- **Standard Hooks and Headed Deformed Bars**
  - Substantial changes from 318-14

Development Length

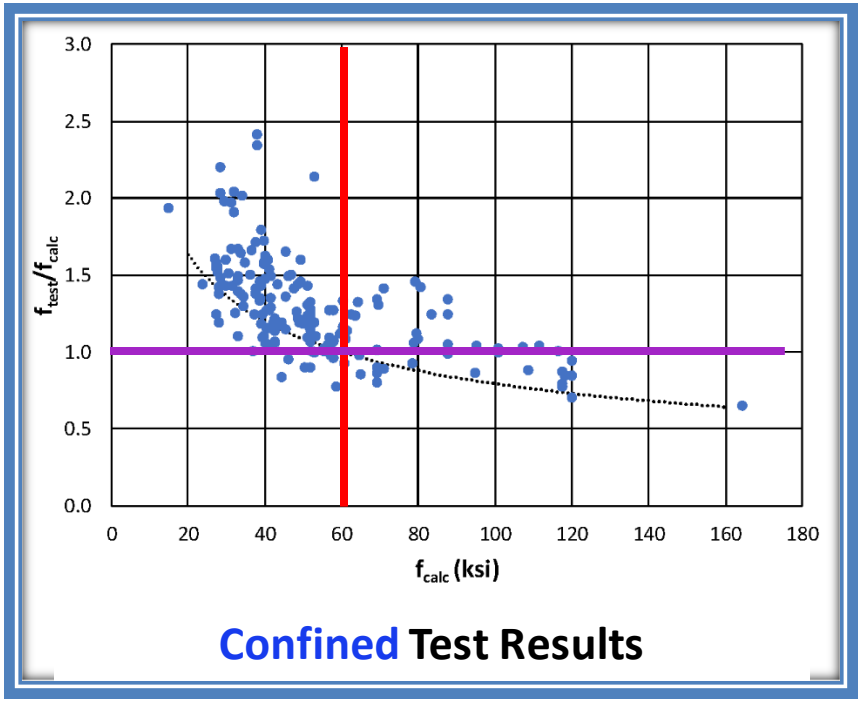
- **Deformed Bars and Deformed Wires in Tension**
- **Standard Hooks in Tension**
- **Headed Deformed Bars in Tension**



Development Length of Deformed Bars and Deformed Wires in Tension



Unconfined Test Results



Confined Test Results

f_{test} = reinforcement stress at the time of failure

f_{calc} = calculated stress by solving ACI 318-14 Equation 25.4.2.3a

Development Length of Deformed Bars and Deformed Wires in Tension

- Modification in simplified provisions of 25.4.2.3
- Ψg : new modification factor based on grade of reinforcement
- Modification in Table 25.4.2.3

Table 25.4.2.3—Development length for deformed bars and deformed wires in tension

Spacing and cover	No. 6 and smaller bars and deformed wires	No. 7 and larger bars
Clear spacing of bars or wires being developed or lap spliced not less than d_b, clear cover at least d_b, and stirrups or ties throughout ℓ_d not less than the Code minimum or Clear spacing of bars or wires being developed or lap spliced at least $2d_b$ and clear cover at least d_b	$\left(\frac{f_y \Psi_t \Psi_e \Psi_g}{25\lambda\sqrt{f'_c}}\right) d_b$	$\left(\frac{f_y \Psi_t \Psi_e \Psi_g}{20\lambda\sqrt{f'_c}}\right) d_b$
Other cases	$\left(\frac{3 f_y \Psi_t \Psi_e \Psi_g}{50\lambda\sqrt{f'_c}}\right) d_b$	$\left(\frac{3 f_y \Psi_t \Psi_e \Psi_g}{40\lambda\sqrt{f'_c}}\right) d_b$

Development Length of Deformed Bars and Deformed Wires in Tension

- **Modification in general development length equation 25.4.2.4(a)**

$$\ell_d = \left(\frac{3}{40} \frac{f_y}{\lambda \sqrt{f_c'}} \frac{\psi_t \psi_e \psi_s \psi_g}{\left(\frac{c_b + K_{tr}}{d_b} \right)} \right) d_b$$

Modification factors

λ : Lightweight

ψ_t : Casting position

ψ_e : Epoxy

ψ_s : Size

ψ_g : Reinforcement grade

- **Provision 25.4.2.2**

  $K_{tr} \geq 0.5 d_b$ for $f_y \geq 80{,}000$ **psi** , if longitudinal bar spacing **< 6 in**.

Development Length of Deformed Bars and Deformed Wires in Tension

Table 25.4.2.5—Modification factors for development of deformed bars and deformed wires in tension

Modification factor	Condition	Value of factor
Lightweight λ	Lightweight concrete	0.75
	Normalweight concrete	1.0
Reinforcement grade ψ_g	Grade 40 or Grade 60	1.0
	Grade 80	1.15
	Grade 100	1.3
Epoxy[1] ψ_e	Epoxy-coated or zinc and epoxy dual-coated reinforcement with clear cover less than $3d_b$ or clear spacing less than $6d_b$	1.5
	Epoxy-coated or zinc and epoxy dual-coated reinforcement for all other conditions	1.2
	Uncoated or zinc-coated (galvanized) reinforcement	1.0
Size ψ_s	No. 7 and larger bars	1.0
	No. 6 and smaller bars and deformed wires	0.8
Casting position[1] ψ_t	More than 12 in. of fresh concrete placed below horizontal reinforcement	1.3
	Other	1.0

Example—Development Length of Deformed Bars and Deformed Wires in Tension

Check development length of **No. 8** longitudinal bar in a beam. Assume f'_c **= 4000 psi NWC**, **Grade 80** reinforcement, 2 in. cover and **no epoxy coating**.

$$\ell_d = \left(\frac{3}{40} \frac{f_y}{\lambda \sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s \psi_g}{\left(\frac{c_b + K_{tr}}{d_b} \right)} \right) d_b$$

λ = 1.0

ψ_e = 1.0

ψ_s = 1.0

ψ_t = 1.0

$\psi_t \psi_e$ = 1.0 < 1.7

ψ_g = 1.15

From Table 25.4.2.5

ψ_g	Grade 40 or Grade 60	1.0
	Grade 80	1.15
	Grade 100	1.3

confinement term **$(c_b + K_{tr})/d_b$ = 2.5** (using the upper limit)

Example—Development Length of Deformed Bars and Deformed Wires in Tension

Substituting in Eq. 25.4.2.4a:

$$\ell_d = \left(\frac{3}{40}\frac{80{,}000}{(1)\sqrt{4000}}\frac{(1)(1)(1)(1.15)}{2.5}\right)(1.0) = 43.6 \text{ in.}$$

In comparison a similar bar with ***Grade 60*** reinforcement;

$$\ell_d = \left(\frac{3}{40}\frac{60{,}000}{(1)\sqrt{4000}}\frac{(1)(1)(1)(1)}{2.5}\right)(1.0) = 28.5 \text{ in.}$$

Increase of ~ **50 percent** in development length for Grade 80

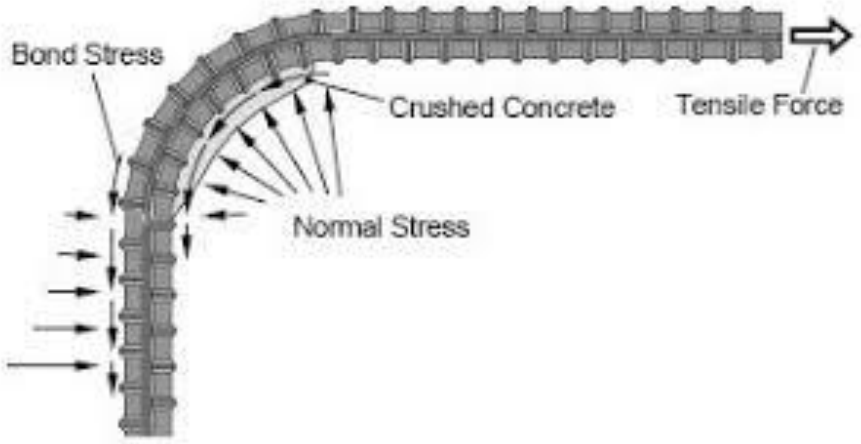
Development Length of Deformed Bars and Deformed Wires in Tension

- Differences in higher grade steel for 4000 psi concrete

Grade	ψ_g	$\ell_{d,Gr\#}/\ell_{d,Gr60}$
60	1.0	1.0
80	1.15	1.5
100	1.3	2.2

Development Length

- Deformed Bars and Deformed Wires in Tension
- **Standard Hooks in Tension**
- Headed Deformed Bars in Tension



Development Length of Std. Hooks in Tension

- **Failure Modes**



Front Pullout



Front Blowout



Side splitting



Side blowout

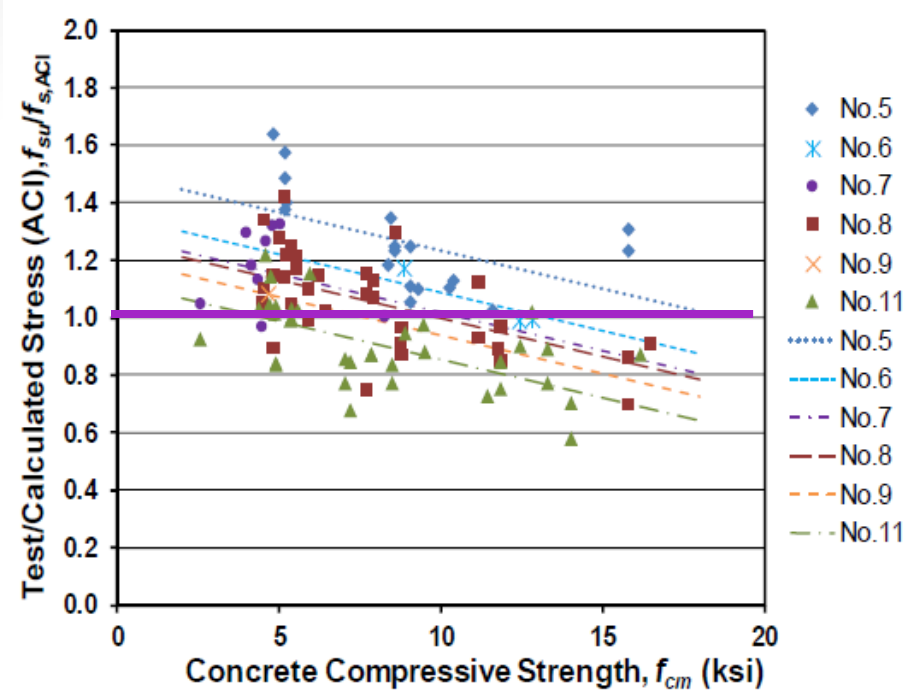


Tail kickout

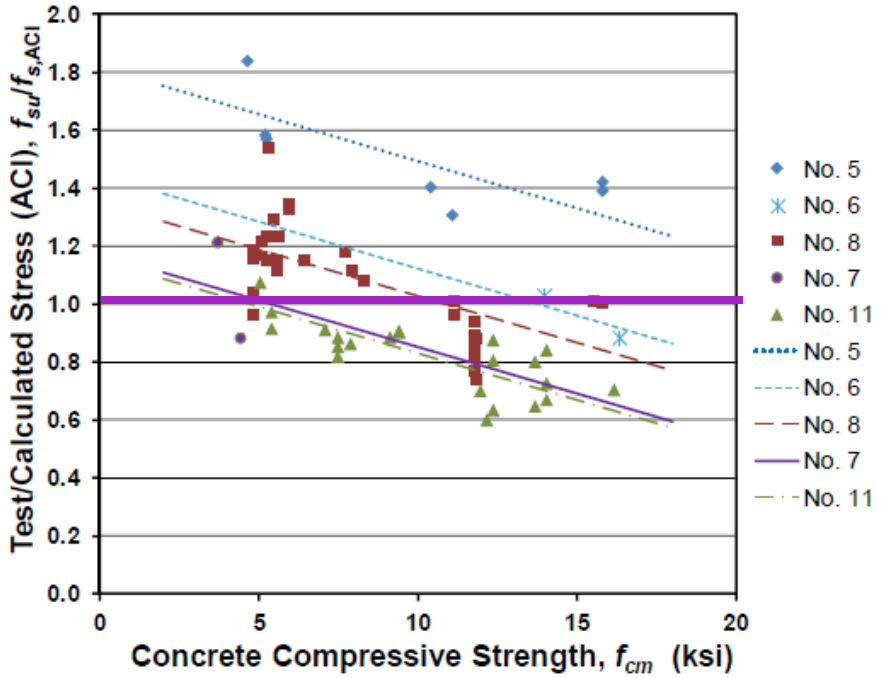
- **Mostly, front and side failures**
  - Dominant front failure (pullout and blowout)
  - **Blowouts** were more sudden in nature

Development Length of Std. Hooks in Tension

$$ACI\ 318-14:\ \ell_{dh} = \left(\frac{f_y \psi_e \boldsymbol{\psi_c} \boldsymbol{\psi_r}}{50\lambda\sqrt{f_c'}}\right) d_b$$



Unconfined Test Results



Confined Test Results

f_{SU} = stress at anchorage failure for the hooked bar

$f_{s,ACI}$ = stress predicted by the ACI development length equation

Development Length of Std. Hooks in Tension

- **25.4.3.1**—Development length of standard hooks in tension is the greater of (a) through (c):

(a) $\left(\frac{f_y \psi_e \psi_r \psi_o \psi_c}{55\lambda\sqrt{f'_c}}\right) d_b^{1.5}$

(b) $8d_b$

(c) 6 in

ACI 318- 14

$$\ell_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50\lambda\sqrt{f'_c}}\right) d_b$$

- Modification factors
  - ψ_r : Confining reinforcement (**redefined)**
  - ψ_o : Location **(new)**
  - ψ_c : Concrete strength (**new** – used for cover in the past)

Development Length of Std. Hooks in Tension

Table 25.4.3.2: Modification factors for development of hooked bars in tension

Modification factor	Condition	Value of factor
318-14 Confining reinforcement, ψ_r	For 90-degree hooks of No. 11 and smaller bars (1) enclosed along ℓ_{dh} within ties or stirrups perpendicular to ℓ_{dh} at $\mathbf{s \le 3d_b}$, or (2) enclosed along the bar extension **beyond hook** including the bend within ties or stirrups perpendicular to ℓ_{ext} at $\mathbf{s \le 3d_b}$	0.8
	Other	1.0
318-19 Confining reinforcement, ψ_r	For No.11 and smaller bars with $A_{th} \ge 0.4A_{hs}$ or $s \ge 6d_b$	1.0
	Other	1.6

Development Length of Std. Hooks in Tension

25.4.3.3:

- **Confining reinforcement (A_{th}) shall consists of (a) or (b)**
  - **(a)** Ties or stirrups that enclose the hook and satisfy 25.3.2
  - **(b)** Other reinf. that extends at least **$0.75\ell_{dh}$** from the enclosed hook in the direction of the bar in tension and in accordance with (1) or (2)
    - parallel or perpendicular (Fig. R25.4.3.3a and Fig. R25.4.3.3b)

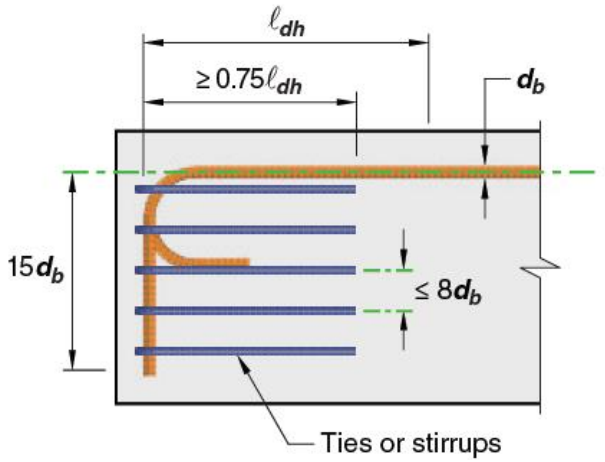


Fig. R25.4.3.3a

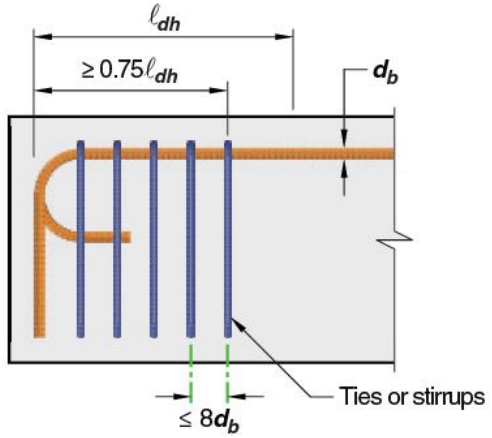


Fig. R25.4.3.3b

Development Length of Std. Hooks in Tension

- **(1) Confining reinforcement placed parallel to the bar** (Typical in beam-column joint)
  - **Two or more** ties or stirrups parallel to ℓ_{dh} enclosing the hooks
  - Evenly distributed with a center-to-center **spacing** $\leq 8d_b$
  - within $15d_b$ **of the centerline** of the straight portion of the hooked bars

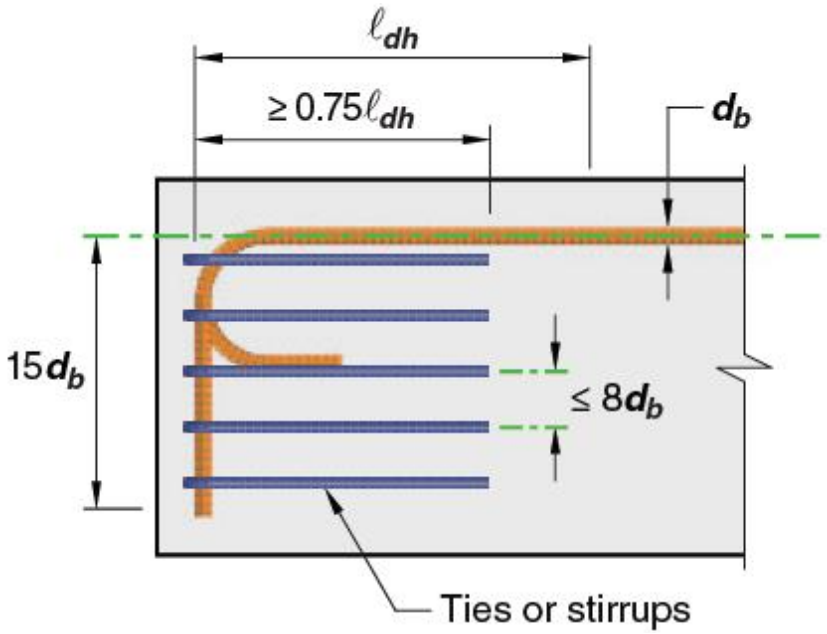


Fig. R25.4.3.3a

Development Length of Std. Hooks in Tension

- **(2) Confining reinforcement placed perpendicular to the bar**
  - **Two or more** ties or stirrups perpendicular to ℓ_{dh} enclosing the hooks
  - Evenly distributed with a center-to-center spacing **≤ 8db**

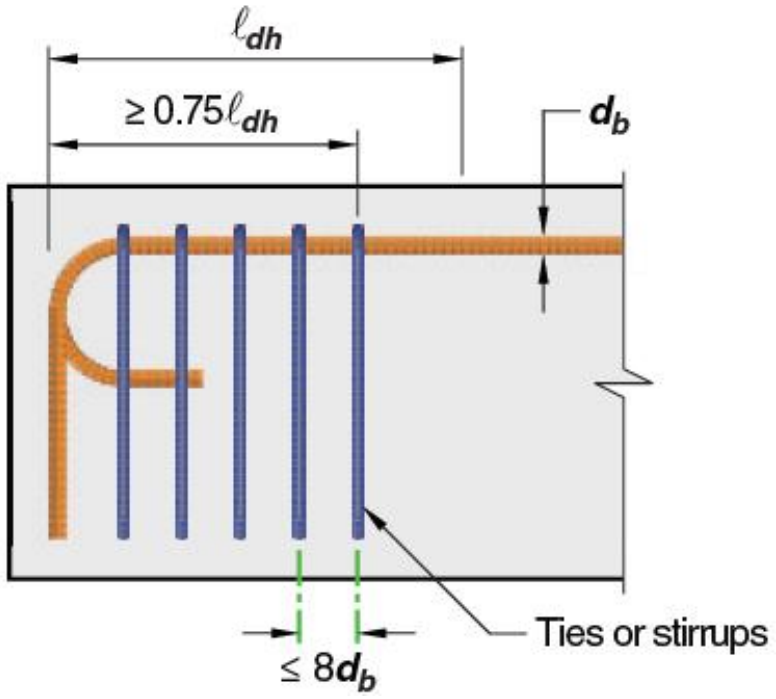


Fig. R25.4.3.3b

Development Length of Std. Hooks in Tension

Table 25.4.3.2: Modification factors for development of hooked bars in tension

Modification factor	Condition	Value of factor
318-14 Cover ψ_c	For No. 11 bar and smaller hooks with side cover (normal to plane of hook) ≥ 2-1/2 in. and for 90-degree hook with cover on bar extension beyond hook ≥ 2 in.	0.7
	Other	1.0
318-19 Location, ψ_o	For No.11 and smaller diameter hooked bars (1) Terminating inside column core w/ side cover normal to plane of hook ≥ 2.5 in., or (2) with side cover normal to plane of hook ≥ $6d_b$	1.0
	Other	1.25

Development Length of Std. Hooks in Tension

Table 25.4.3.2: Modification factors for development of hooked bars in tension

Modification factor	Condition	Value of factor
Concrete strength, ψ_c	For $f'_c < 6000$ psi	$f'_c/15{,}000 + 0.6$
	For $f'_c \geq 6000$ psi	1.0

Example—Development Length of Std Hook

Check hooked bar anchorage of longitudinal beam reinforcement, **3-No. 10** bars in a **20 x 20 in.** exterior column. Assume **f'_c = 4000 psi NWC**, **Grade 60** reinforcement, **2.5 in. cover** normal to plane of hook, and no epoxy coating. Steel confinement is provided such that $A_{th} = 0.4\,A_{hs}$.

$$\ell_{dh} = \left(\frac{f_y \psi_e \psi_r \psi_o \psi_c}{55\lambda\sqrt{f'_c}} \right) d_b^{1.5}$$

λ = 1.0
ψ_e = 1.0
ψ_r = 1.0
ψ_o = 1.0
ψ_c = $f'_c/15{,}000 + 0.6 = 4{,}000/15{,}000 + 0.6$ **= 0.87**

Example—Development Length of Std Hook

Substituting in the equation:

$$\ell_{dh} = \left(\frac{(60{,}000)(1.0)(1.0)(1.0)(0.87)}{(55)(1.0)\sqrt{4{,}000}}\right)(1.27)^{1.5}$$

ℓ_{dh} = 21.5 in. > 20 in. **NG**

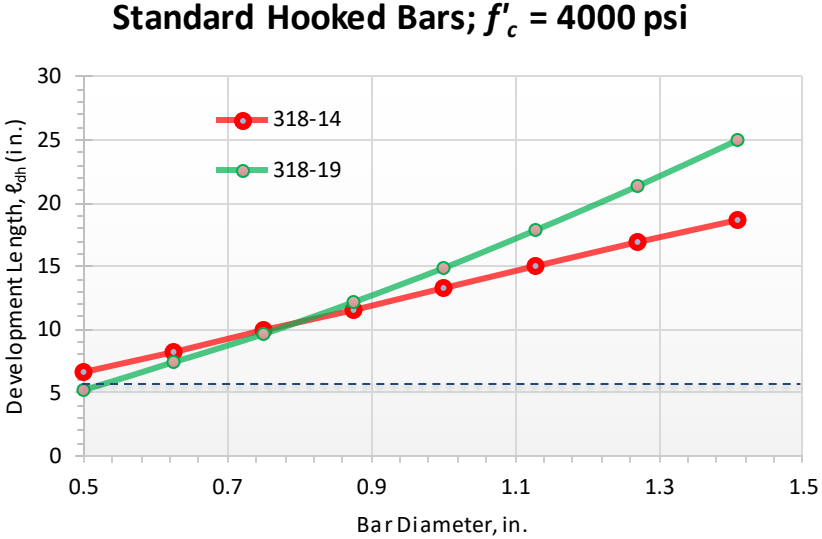
In comparison to the equation in **318-14:**

$$\ell_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50\lambda\sqrt{f_c'}}\right) d_b$$

ψ_e = 1.0
ψ_c = 0.7 (2 -1/2 in. side cover and 2 in. back cover)
ψ_r = 1.0

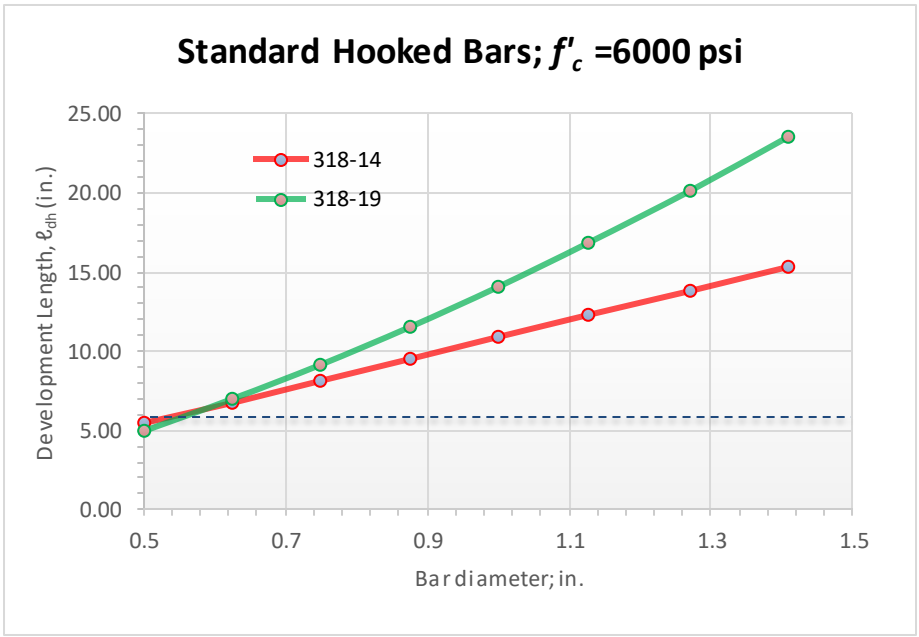
$\ell_{dh(318\text{-}14)}$ = 16.9 in. < 20 in. **OK**

Example—Development Length of Std Hook



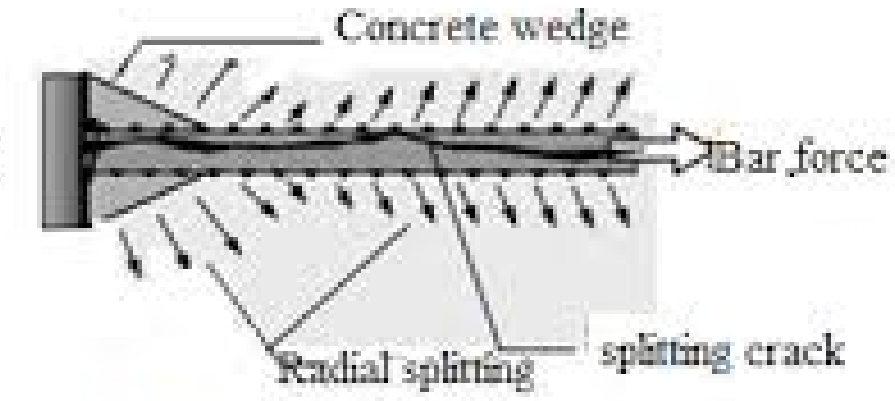
$$\ell_{dh} = \left(\frac{f_y \psi_e \psi_r \psi_o \psi_c}{55\lambda\sqrt{f'_c}} \right) d_b^{1.5}$$

$$\ell_{dh} = \left(\frac{f_y \psi_e \psi_c \psi_r}{50\lambda\sqrt{f'_c}} \right) d_b$$



Development Length

- Deformed Bars and Deformed Wires in Tension
- Standard Hooks in Tension
- **Headed Deformed Bars in Tension**



Development Length of Headed Deformed Bars in Tension

25.4.4.1 Use of a head to develop a deformed bar in tension shall be permitted if conditions (a) through (f) are satisfied:

(a) Bar shall conform to 20.2.1.6

~~(b) Bar f_y shall not exceed 60,000 psi~~

(b) Bar size shall not exceed No. 11

(c) Net bearing area of head $\boldsymbol{A_{brg}}$ shall be at least $\boldsymbol{4A_b}$

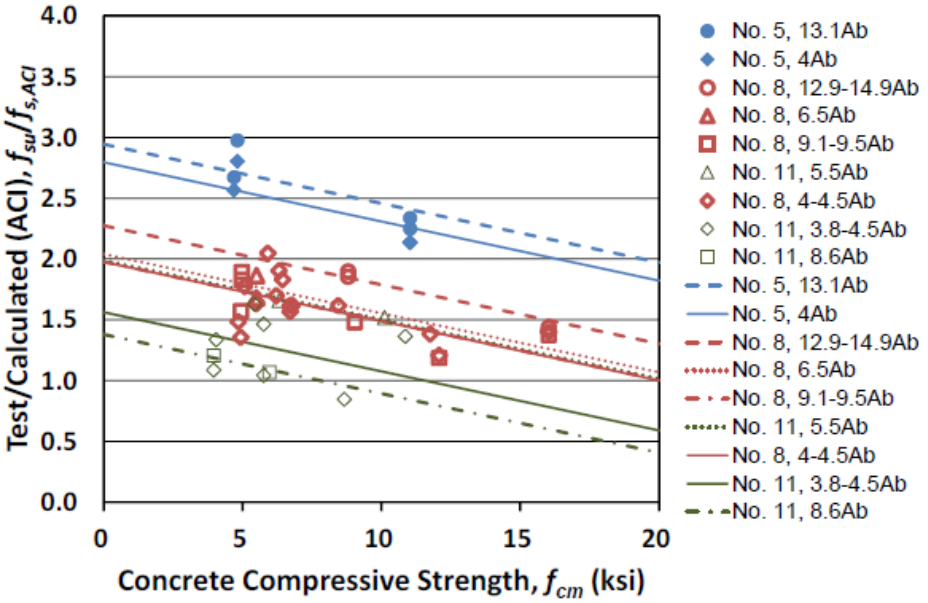
(d) Concrete shall be normalweight

(e) Clear cover for bar shall be at least $\boldsymbol{2d_b}$

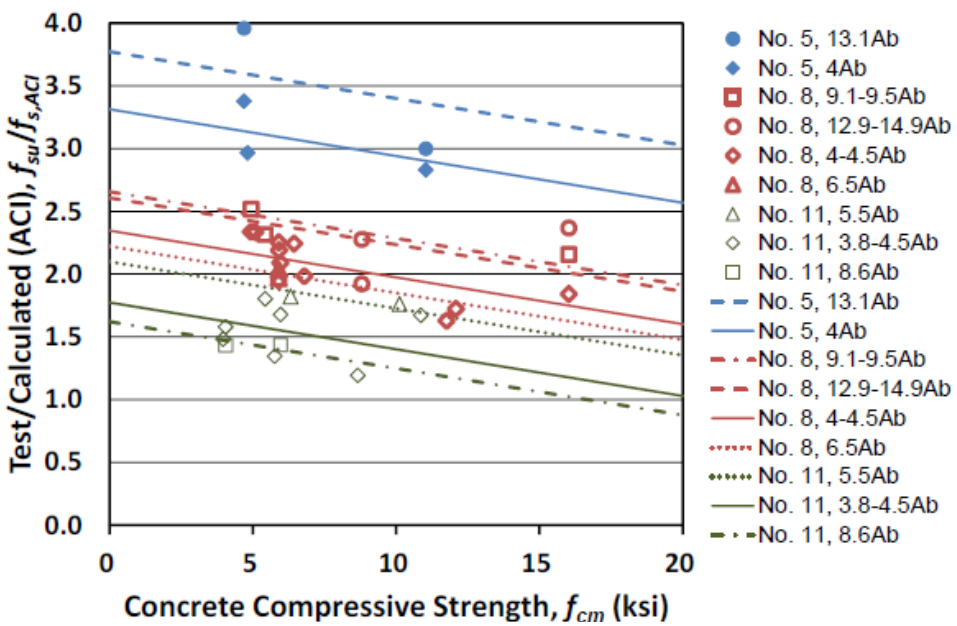
(f) Center-to-center spacing between bars shall be at least $\boldsymbol{3d_b}$

Development Length of Headed Deformed Bars in Tension

$$ACI\ 318-14: \quad \ell_{dt} = \left(\frac{0.016 f_y \psi_e}{\sqrt{f_c'}}\right) d_b$$



Unconfined Test Results



Confined Test Results

f_{SU} = stress at anchorage failure for the hooked bar

$f_{s,ACI}$ = stress predicted by the ACI development length equation

Development Length of Headed Deformed Bars in Tension

- **25.4.4.2**: Development length ℓ_{dt} for headed deformed bars in tension shall be the longest of (a) through (c):

(a) $$\ell_{dt} = \left(\frac{f_y \psi_e \psi_p \psi_o \psi_c}{75\sqrt{f'_c}} \right) d_b^{1.5}$$

(b) $8d_b$

(c) 6 in.

ACI 318- 14

$$\ell_{dt} = \left(\frac{0.016 f_y \psi_e}{\sqrt{f'_c}} \right) d_b$$

$$f'_c \leq 6000\ psi$$

- Modification factors
  - ψ_p : Parallel tie reinforcement
  - ψ_o : Location
  - ψ_c : Concrete strength

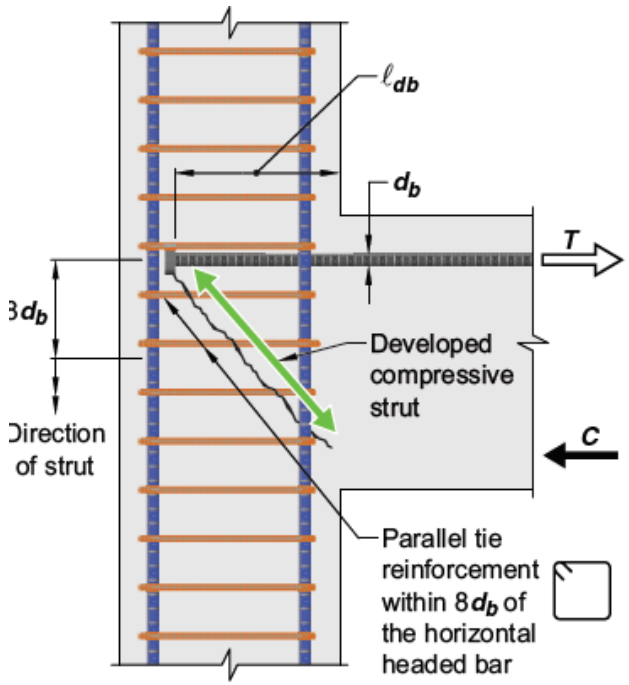
Development length of Headed Deformed Bars in Tension

Table 25.4.4.3—Modification factors for development of headed bars in tension

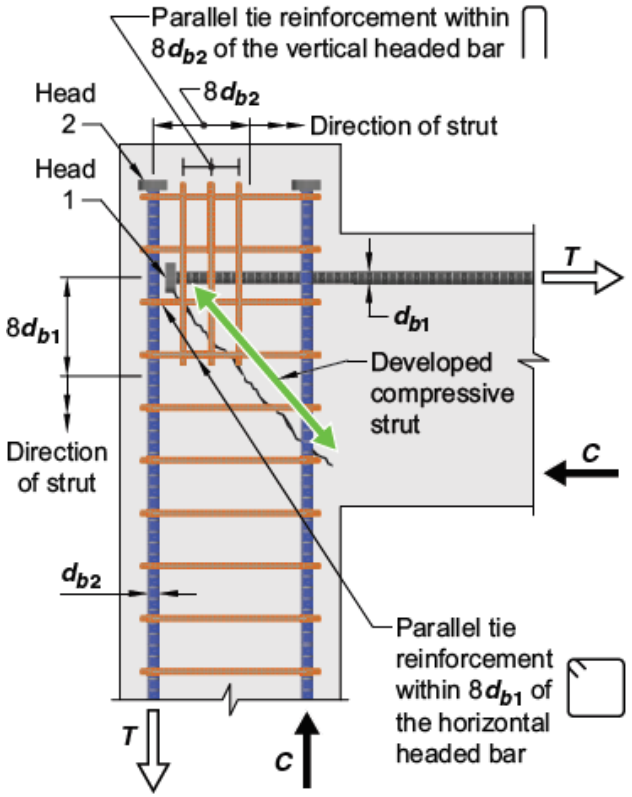
Modification factor	Condition	Value of factor
Parallel tie reinforcement, ψ_p	For No.11 and smaller bars with $A_{tt} \geq 0.3A_{hs}$ or $s \geq 6d_b$	1.0
	Other	1.6
Location, ψ_o	For headed bars (1) Terminating inside column core w/ side cover to bar ≥ 2.5 in., or (2) with side cover to bar ≥ $6d_b$	1.0
	Others	1.25
Concrete strength, ψ_c	For $f'_c < 6000$ psi	$f'_c/15{,}000+0.6$
	For $f'_c \geq 6000$ psi	1.0

Development Length of Headed Deformed Bars in Tension

- **Parallel tie reinforcement (A_{tt})**
  - locate within **$8d_b$** of the centerline of the headed bar toward the middle of the joint



(a) Horizontal headed bars



(b) Vertical and horizontal headed bars

Example—Development Length of Headed Deformed Bars in Tension

Check development length of **No. 9** longitudinal bar in a beam. Assume f'_c **= 4000 psi NWC**, **Grade 60** reinforcement, **2.5 in. cover,** and no epoxy coating. Steel confinement is provided such that $A_{tt} = 0.3\,A_{hs}$.

$$\ell_{dt} = \left(\frac{f_y \psi_e \psi_p \psi_o \psi_c}{75\sqrt{f'_c}} \right) d_b^{1.5}$$

ψ_e **= 1.0**

ψ_p **= 1.0**

ψ_o **= 1.0**

ψ_c **=** $f'_c/15{,}000 + 0.6 = 4{,}000/15{,}000+0.6$ **= 0.87**

Example—Development Length of Headed Deformed Bars in Tension

Substituting in the equation :

$$\ell_{dt} = \left(\frac{(60{,}000)(1.0)(1.0)(1.0)(0.87)}{(75)\sqrt{4{,}000}}\right)(1.128)^{1.5}$$

ℓ_{dt} = 13.2 in.

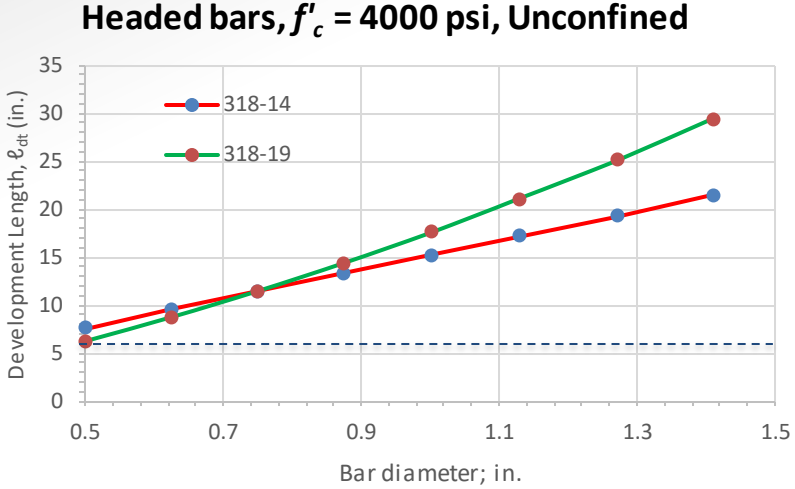
In comparison to the equation in **318-14:**

$$\ell_{dt} = \left(\frac{0.016(1.0)(60{,}000)}{\sqrt{4{,}000}}\right)(1.128)$$

$\ell_{dt(318\text{-}14)}$ = 17.1 in.

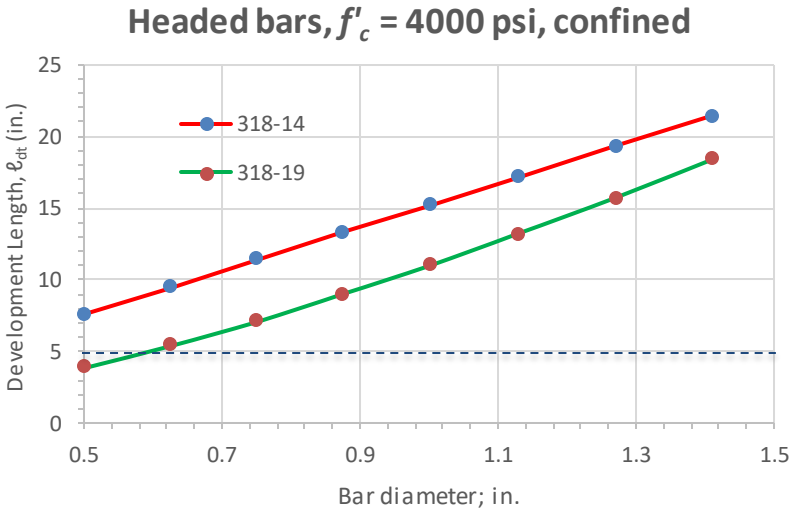
- Decrease in development length of headed bars in tension as per 318-19 in this example
  - No.11 and smaller bars with A_{tt} $0.3A_{ts}$
  - bars terminating inside column core with side cover to bar ≥ 2.5 in

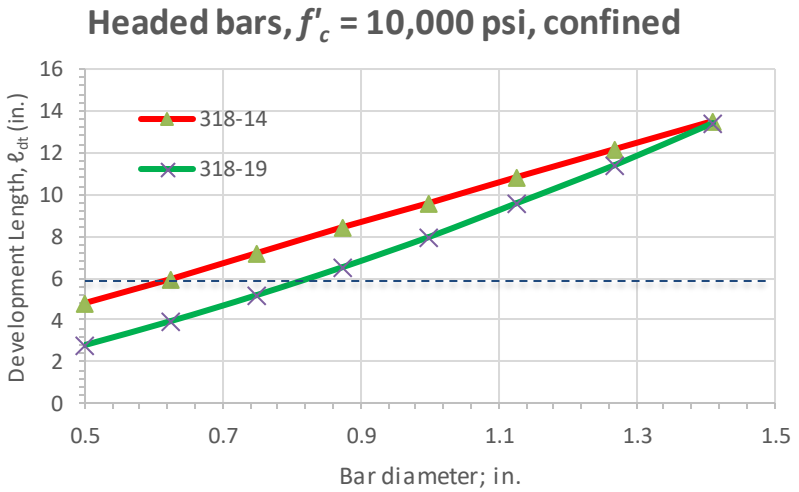
Example—Development Length of Headed Deformed Bars in Tension



$$\ell_{dt} = \left(\frac{f_y \psi_e \psi_p \psi_o \psi_c}{75\sqrt{f'_c}} \right) d_b^{1.5}$$

$$\ell_{dt} = \left(\frac{0.016 f_y \psi_e}{\sqrt{f'_c}} \right) d_b$$





ACI 318-19

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Shear Modifications



Shear equations change

- **One-way beam/slab shear – provision 22.5**
  - Size effect
  - Reinforcement ratio
- **Two-way slab shear – provision 22.6**
  - Size effect
  - Reinforcement ratio



Why shear equations changed in 318-19

- **Reasons for changes**
  - Evidence shows
    - Size effect
    - Low ρ_w effect
- **More prevalent**
  - Deeper beams
  - Deep transfer slabs



Other shear changes

- **Wall shear equations**
  - Chapter 11 now similar to Chapter 18
- **Shear leg spacing**
  - Section spacing requirements
- **Biaxial shear**
  - Engineer must consider
- **Hanger reinforcement**
  - Commentary suggestion

ACI 318-19

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One-way Shear Equations

Why one-way shear equations changed in 318-19

- **ACI 445, Shear and Torsion**
  - Four databases vetted and checked

Beam types in database	Number of samples
Reinforced concrete w/o min shear reinforcement	784
Reinforced concrete with min. shear reinforcement	170
Prestressed concrete w/o min. shear reinforcement	214
Prestressed concrete with min. shear reinforcement	117
Total samples	1285

Why one-way shear equations changed in 318-19

d = 10 in. – λ_s, size effect factor

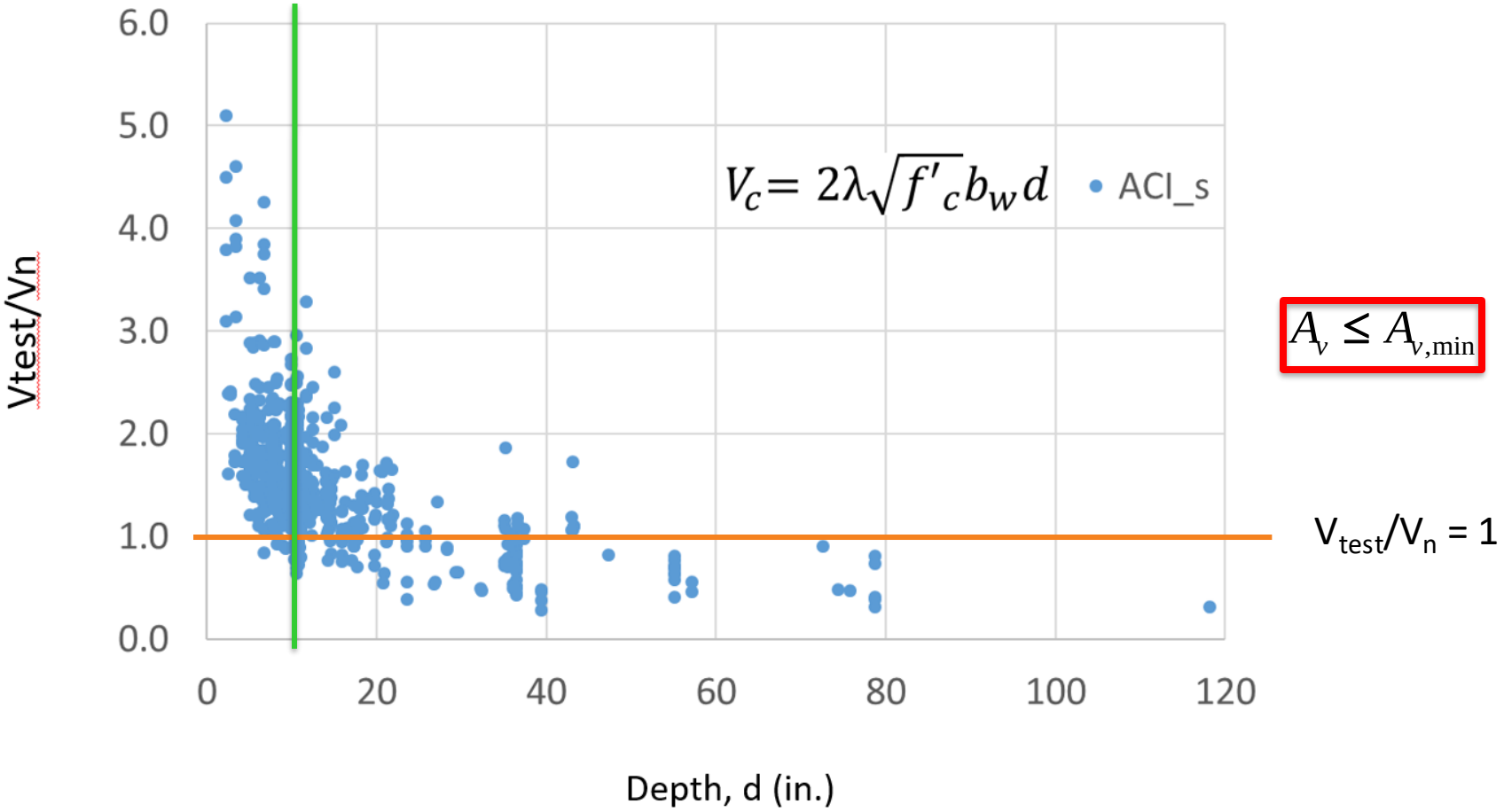


Figure: Strength Ratio (V_{test}/V_n) that was calculated by 318-14 Simplified

Why one-way shear equations changed in 318-19

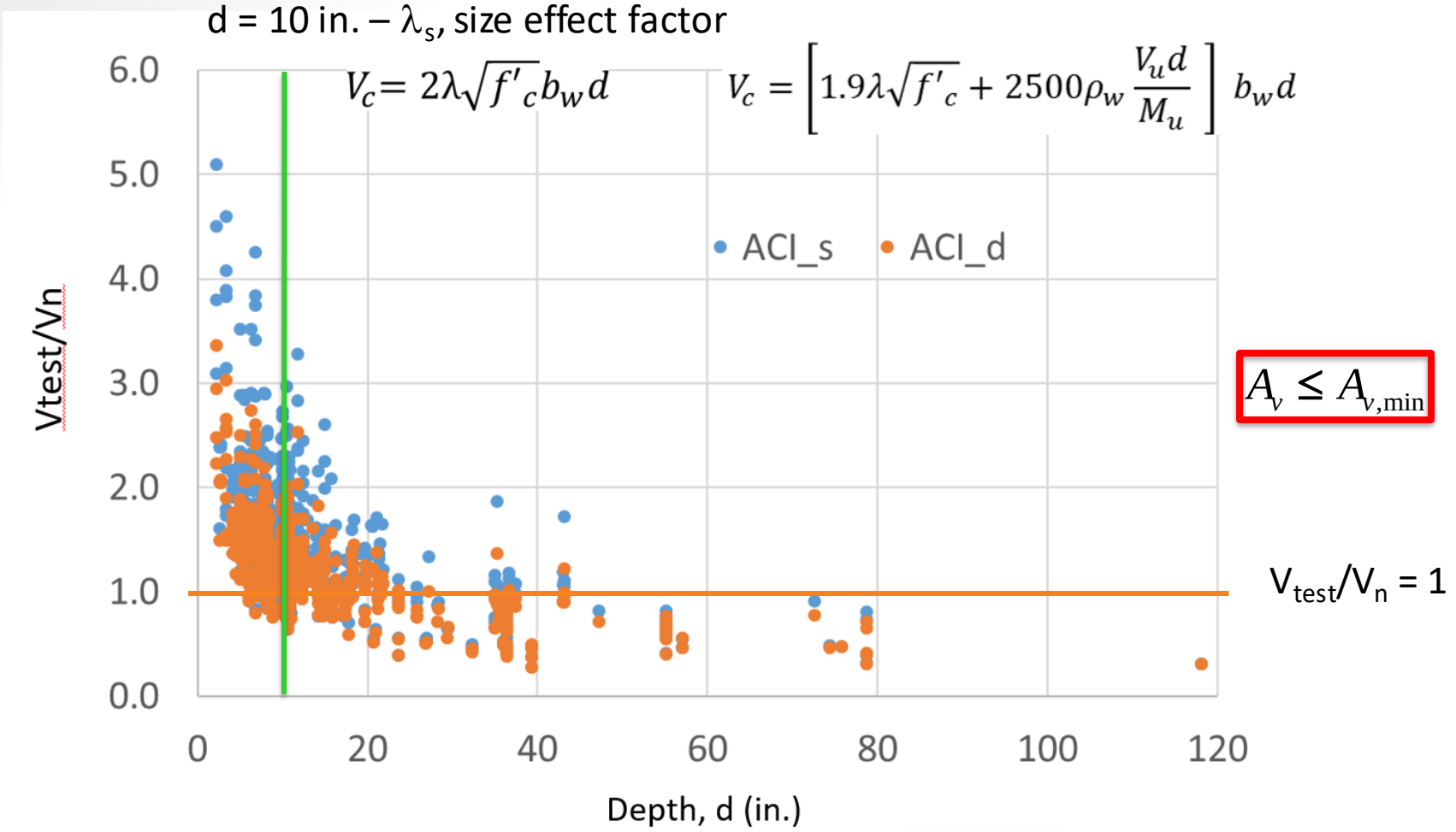


Figure: Strength Ratio (V_{test}/V_n) that was calculated by both ACI 318-14 Simplified and Detailed

Why one-way shear equations changed in 318-19

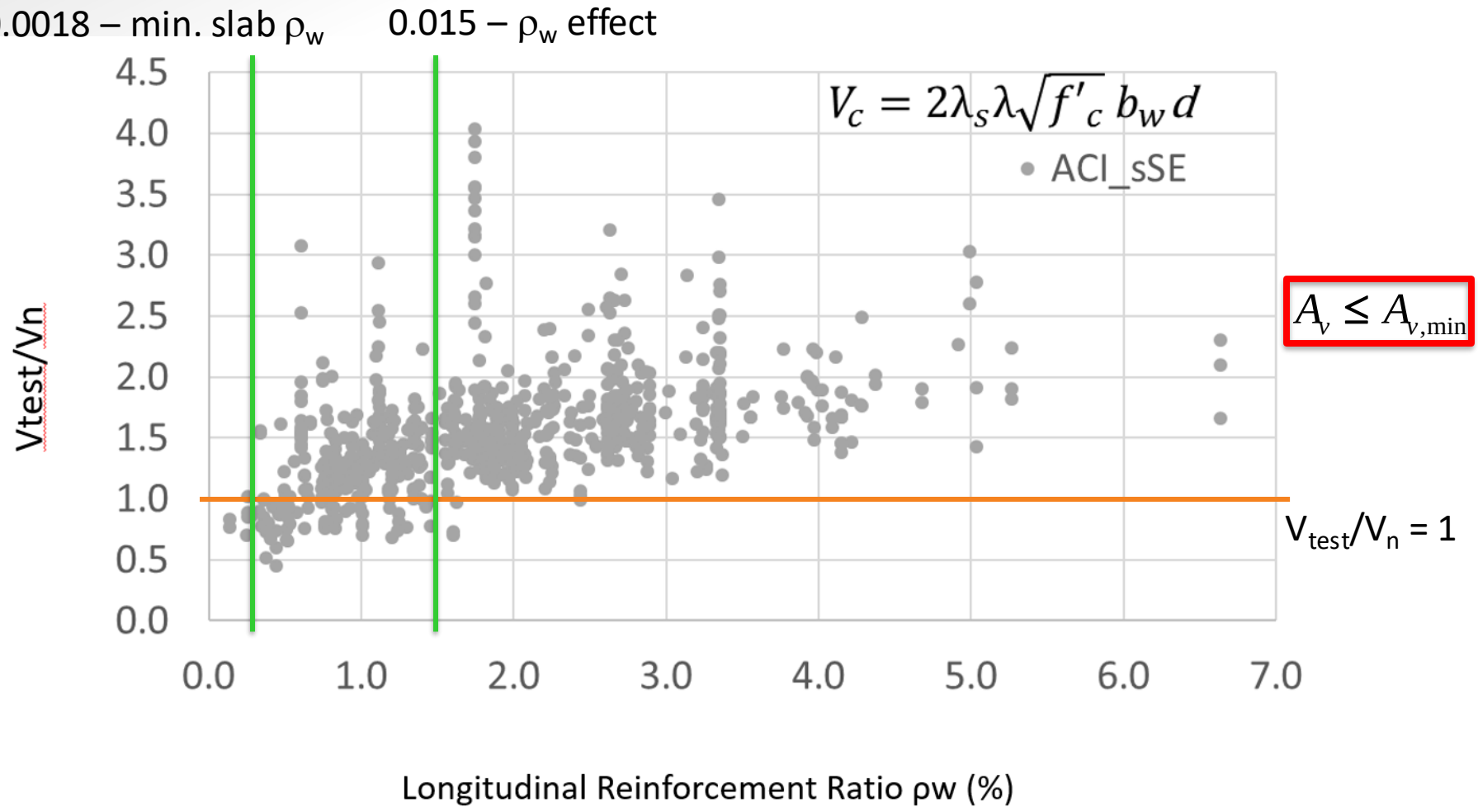


Figure: Strength Ratio (V_{test}/V_n) that was calculated by the Simplified Method of ACI318-19 including size effect

Why one-way shear equations changed in 318-19

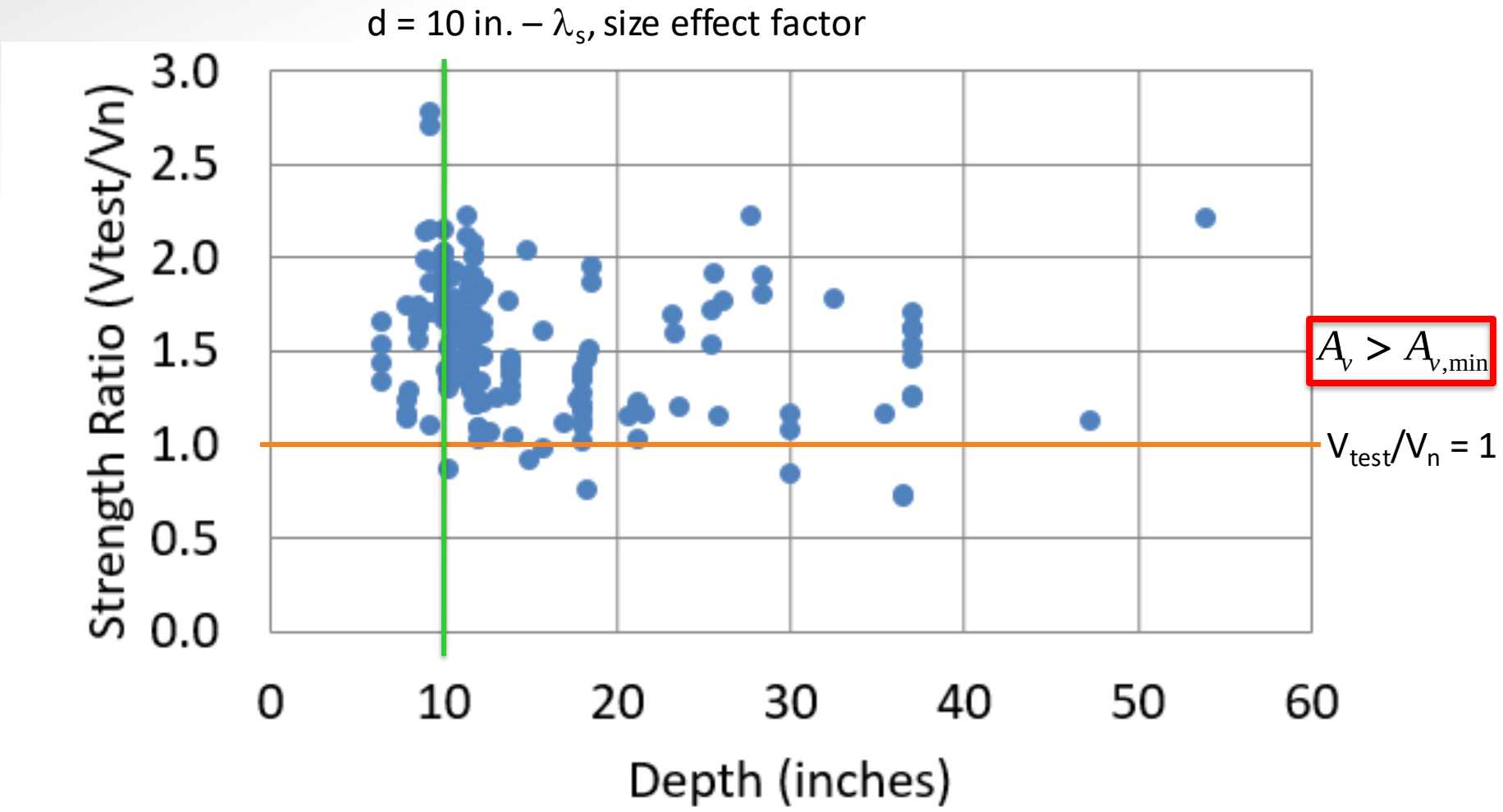


Figure: Strength Ratio (V_{test}/V_n) that was calculated by the Simplified Method of ACI 318-14



Why one-way shear equations changed in 318-19

- **Six different proposals considered**
  - Proposals vetted and considered by
    - ACI 445
    - ACI 318 Subcommittee
    - Public discussion
    - Concrete International articles
- **ACI 318 selected one proposal**

Initial one-way shear provision: goals

- **Include nonprestressed and prestressed**
- **Include axial loading and size effect**
- **Include effect of (ρ_w)**
- **Continue to be proportional to $\sqrt{f'_c}$**
- **And simple**
  - Reduce total number of shear equations
  - Avoid increase in variables
  - Easy to use

Initial one-way shear provision: issues

- **Initial proposal had issues**
  - Unified expressions ≠ V_{ci}, V_{cw}
  - What happened to "2 $\sqrt{f'_c}$"???

Criteria	V_c	
$A_v \geq A_{v,min}$ or $\rho_w + \rho_p \geq 0.015$	$\left[\sqrt{f'_c} + \left(\frac{N_u}{\quad}\right) \right] b_w d$	(a)
All cases	$\left[\lambda_s \lambda (\rho_w + \rho_p)^{1/3} \sqrt{f'_c} + \left(\frac{\quad}{A_g}\right) \right] b_w d$	(b)

Initial one-way shear provision: goals

- Include nonprestressed ~~and prestressed~~
- Include axial loading and size effect
- Include effect of (ρ)
- Continue to be proportional to $\sqrt{f'_c}$
- And ~~simple~~





ACI 318-19 New one-way shear equations

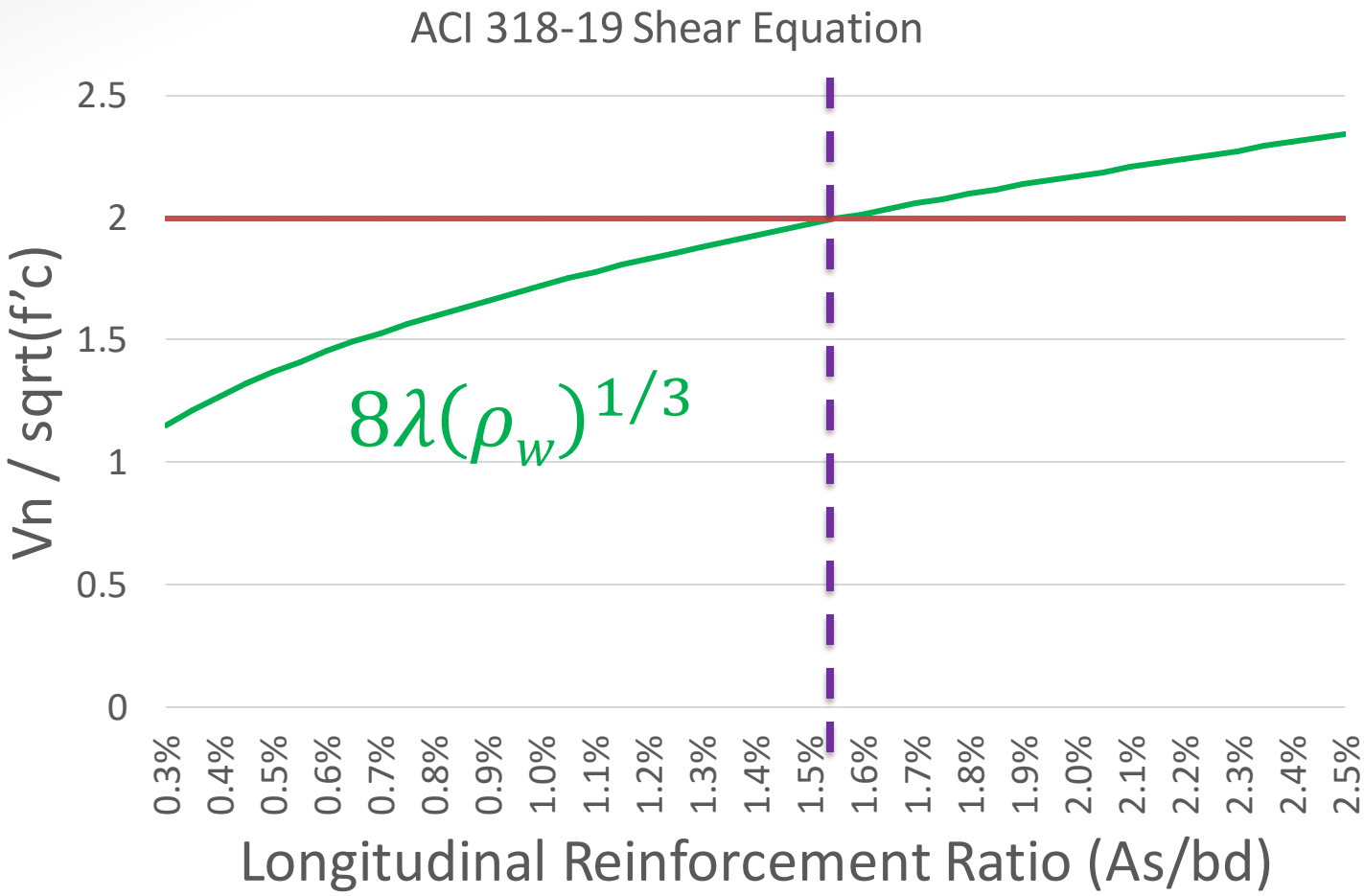
Table 22.5.5.1 - V_c for nonprestressed members

Criteria	V_c		
$A_v \geq A_{v,min}$	Either of:	$\left[2\lambda\sqrt{f'_c} + \frac{N_u}{6A_g}\right] b_w d$	(a)
		$\left[8\lambda(\rho_w)^{1/3}\sqrt{f'_c} + \frac{N_u}{6A_g}\right] b_w d$	(b)
$A_v < A_{v,min}$	$\left[8\lambda_s\lambda(\rho_w)^{1/3}\sqrt{f'_c} + \frac{N_u}{6A_g}\right] b_w d$		(c)

Notes:

1. Axial load, N_u, is positive for compression and negative for tension
2. V_c shall not be taken less than zero.

Effect of ρ_w



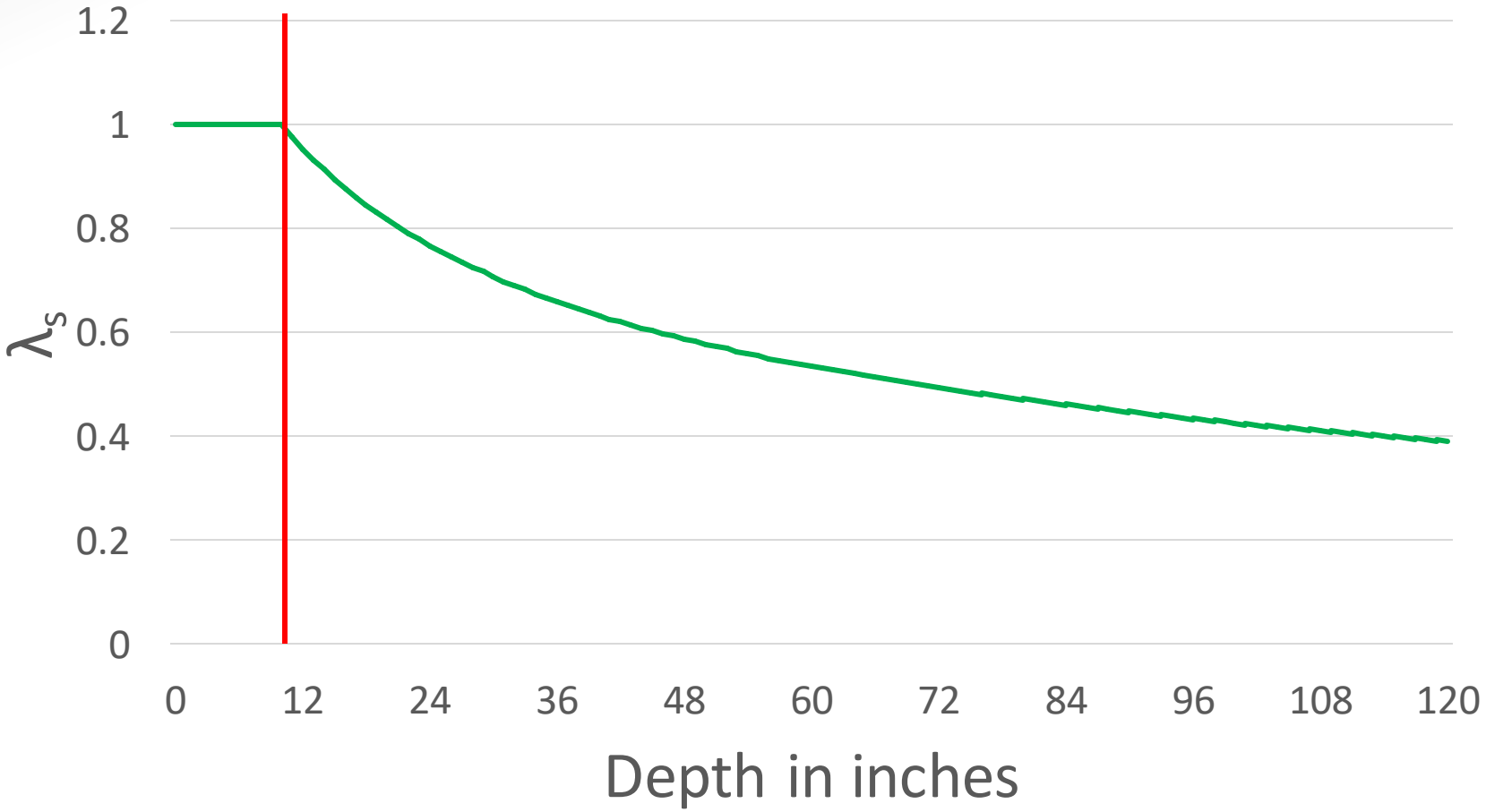
Size effect – what is λ_s?

Provision 22.5.5.1.3 defines λ_s as:

$$\lambda_s = \sqrt{\frac{2}{1+\frac{d}{10}}} \leq 1.0$$

Size effect – what is λ_s?

$$\lambda_s = \sqrt{\frac{2}{1+\frac{d}{10}}} \leq 1.0$$



Other limitations for Table 22.5.5.1

- **Provision 22.5.5.1.1:**
  - Limits the maximum value of V_c

$$V_c \le 5\lambda\sqrt{f_c'}\,b_w d$$

- **Provision 22.5.5.1.2:**
  - Limits the maximum value of the $N_u/6A_g$ term

$$\frac{N_u}{6A_g} \le 0.05\,f_c'$$

9.6.3.1 - Minimum shear reinforcement

- **ACI 318-14**
  - $A_{v,min}$ required if $V_u > 0.5\,\phi V_c$
- **ACI 318-19**
  - $A_{v,min}$ required if $V_u > \phi\lambda\sqrt{f'_c}\,b_w d$
- **Exceptions in Table 9.6.3.1**

22.5.6.2.3—Prestressed members:

Table 22.5.6.2—Approximate method for calculating V_c

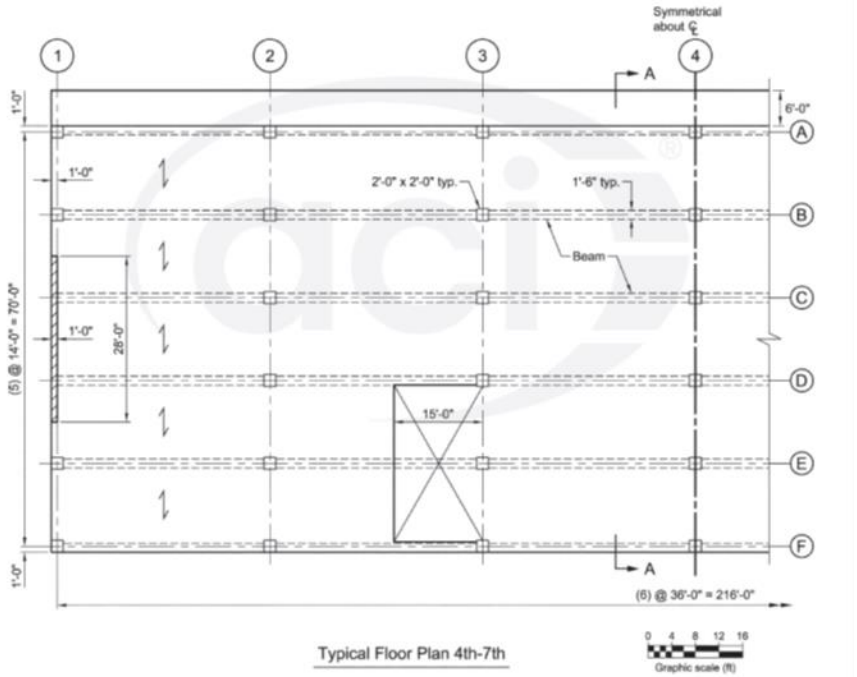
V_c		
Least of (a), (b), and (c):	$\left(0.6\lambda\sqrt{f_c'}+700\frac{V_u d_p}{M_u}\right)b_w d$ [1],[2]	(a)
	$(0.6\lambda\sqrt{f_c'}+700)b_w d$	(b)
	$5\lambda\sqrt{f_c'}b_w d$	(c)

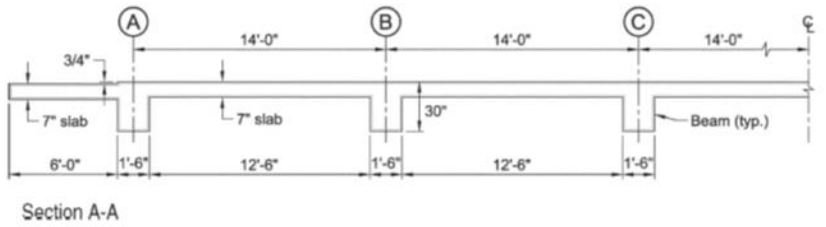
[1] M_u occurs simultaneously with V_u at the section considered.

[2] When calculating the $V_u d_p/M_u$ term in Eq. 22.5.6.2(a), d_p is the distance from the extreme compression fiber to the centroid of prestressed reinforcement. It shall not be permitted to take d_p as **0.80***h* as in 22.5.2.1.

Examples: SP-17(14) 5.7 One-way slab Example 1

- Span = 14 ft
- Live load = 100 psf
- Slab = 7 in. thick
- f'_c = 5000 psi
- No. 5 bars at 12 in.
- d~6 in.
- b = 12 in.
- A_v = 0 in.^{2}
- A_s = 0.31 in.^{2}/ft
- V_u= 2.4 kip/ft





Section A-A

Fig. E1.1—Plan of five-span one-way slab.

Examples: SP-17(14) 5.7 One-way slab Example 1

- SP-17(14) One-way shear calc **ACI 318-14**

$$\phi V_c = \phi 2\lambda \sqrt{f_c'} bd$$

$$\phi V_c = (0.75)(2)(1)\sqrt{5000\,psi}\,(12in.)(6in.)$$

$$\phi V_c = 7.6kip > 2.4kip \therefore OK$$

Examples: SP-17(14) 5.7 One-way slab Example 1

- SP-17(14) One-way shear calc ACI 318-19
- $A_v \le A_{v,min}$, therefore use Eq. 22.5.5.1(c)

$$\phi V_c = \phi 8 \lambda_s \lambda (\rho_w)^{1/3} \sqrt{f_c'} b d$$

$$\rho_w = \frac{0.31}{(12)(6)} = 0.0043 \leftarrow \text{low } \rho_w$$

$$\phi V_c = (0.75)(8)(1)(1)\left(0.0043\right)^{1/3} \sqrt{5000\,psi}\,(12in.)(6in.)$$

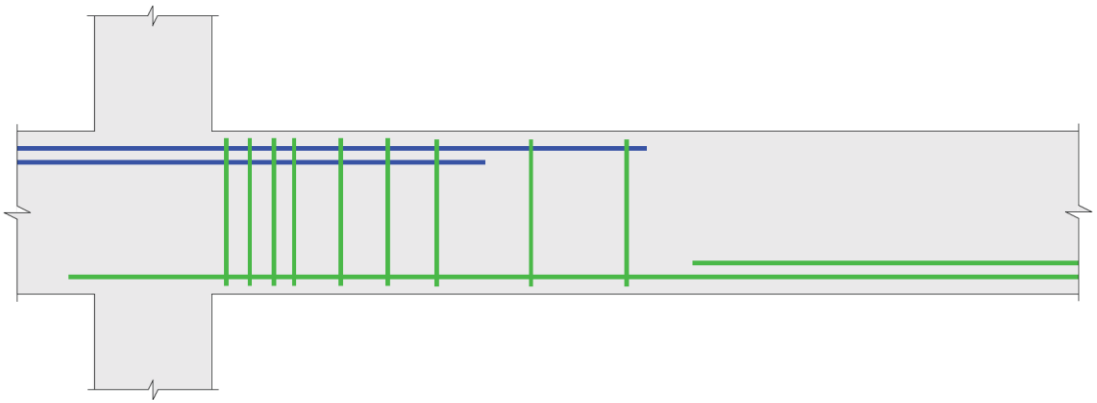
$$\phi V_c = 5.0kip > 2.4kip \therefore OK$$

Examples: SP-17(14) 5.7 One-way slab Example 1

- **ϕV_c ACI 318-19 < ϕV_c ACI 318-14**
  - 318-19 for the example given is ~2/3 of ACI 318-14
  - Effect of low ρ_w
- **Design impact**
  - Thicker slabs if depth was controlled by shear in 318-14.
  - No change if one-way slab thickness was controlled by flexure or deflections

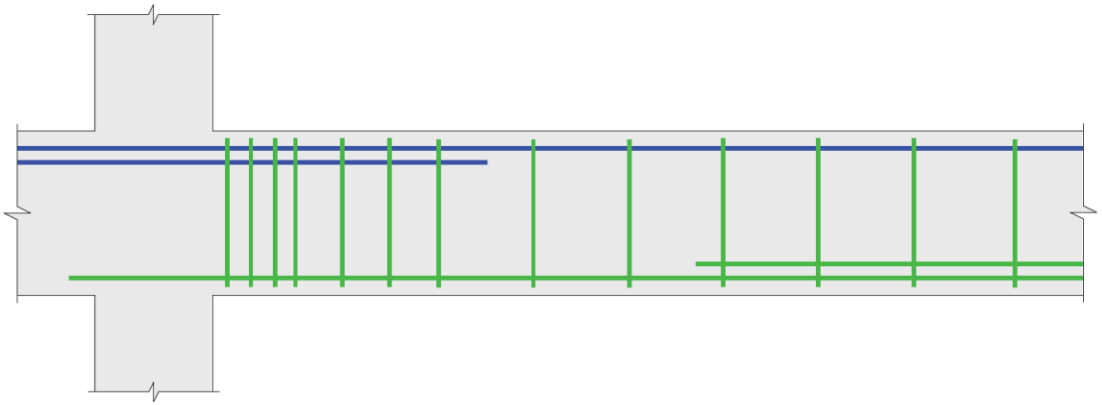
Examples: Beam discussion

- **How many engineers design beams without minimum shear reinforcement?**
- **One-way shear capacity impacted:**
  - $A_{v,min}$ not required and $A_{v,min}$ not used



Examples: Beam discussion

- **Where $A_{v,min}$ installed, Eq. 22.5.5.1(a) $V_c = (2\sqrt{f'_c})$,**
  - ACI 318-14 ~ ACI 318-19
  - Eq. 22.5.5.1(b) of Table 22.5.5.1 permitted
    - $\phi V_c \uparrow \rho_w > 0.015$
- **Provisions encourage $A_{v,min}$**



Examples: SP-17(14) 11.6 Foundation Example 1

- ℓ = 12 ft
- h = 30 in.
- d~25.5 in.
- f'_c = 4000 psi
- 13-No. 8 bars
- b = 12 ft
- A_v = 0 in.^{2}
- A_s = 10.27 in.^{2}
- Analysis V_u= 231 kip

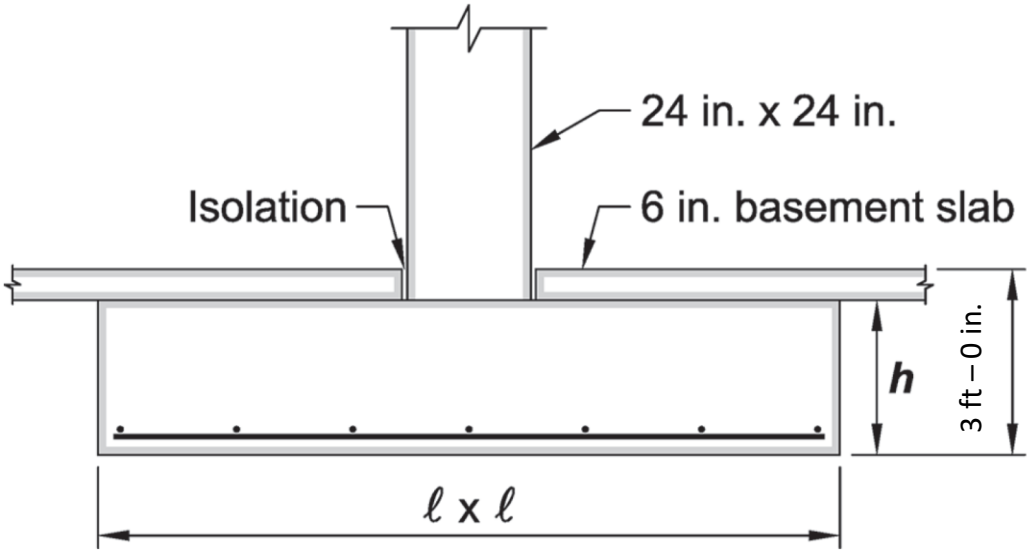


Fig. E1.1—Rectangular foundation plan.

Examples: SP-17(14) 11.6 Foundation Example 1

- SP-17(14) One-way shear calc **ACI 318-14**

$$\phi V_c = \phi 2\lambda \sqrt{f_c'} bd$$

$$\phi V_c = (0.75)(2)(1)\sqrt{4000\,psi}\,(144in.)(25.5in.)$$

$$\phi V_c = 348kip > 231kip \therefore OK$$

Examples: SP-17(14) 11.6 Foundation Example 1

- SP-17(14) One-way shear calc ACI 318-19
- $A_v \leq A_{v,min}$, Eq. 22.5.5.1(c)
- Per ACI 318-19 (13.2.6.2), neglect size effect for:
  - One-way shallow foundations
  - Two-way isolated footings
  - Two-way combined and mat foundations

$$\phi V_c = \phi 8 \lambda (\rho_w)^{1/3} \sqrt{f_c'} bd$$

Examples: SP-17(14) 11.6 Foundation Example 1

- SP-17(14) One-way shear calc ACI 318-19
- $A_v \le A_{v,min}$, Eq. 22.5.5.1(c)

$$\phi V_c = \phi 8 \lambda (\rho_w)^{1/3} \sqrt{f_c'} bd$$

$$\rho_w = \frac{10.27 \text{ in.}^2}{(144 \text{ in.})(25.5 \text{ in.})} = 0.0028$$

$$\phi V_c = (0.75)(8)(1)(0.0028)^{1/3} \sqrt{4000\, psi}\,(144in.)(25.5in.)$$

$$\phi V_c = 196kip < 231kip \therefore NG$$

Examples: SP-17(14) 11.6 Foundation Example 1

- SP-17(14) One-way shear using ACI 318-19
- $A_v \le A_{v,min}$, Eq. 22.5.5.1(c)
- Per ACI 318-19, 13.2.6.2, neglect size effect
- Add 6in. thickness

$$\phi V_c = \phi 8 \lambda (\rho_w)^{1/3} \sqrt{f_c'} bd$$

$$\rho_w = \frac{10.27 \text{ in.}^2}{(144 \text{ in.})(31.5 \text{ in.})} = 0.0023$$

$$\phi V_c = (0.75)(8)(1)(0.0023)^{1/3} \sqrt{4000 \text{ psi}}\,(144 \text{ in.})(31.5 \text{ in.})$$

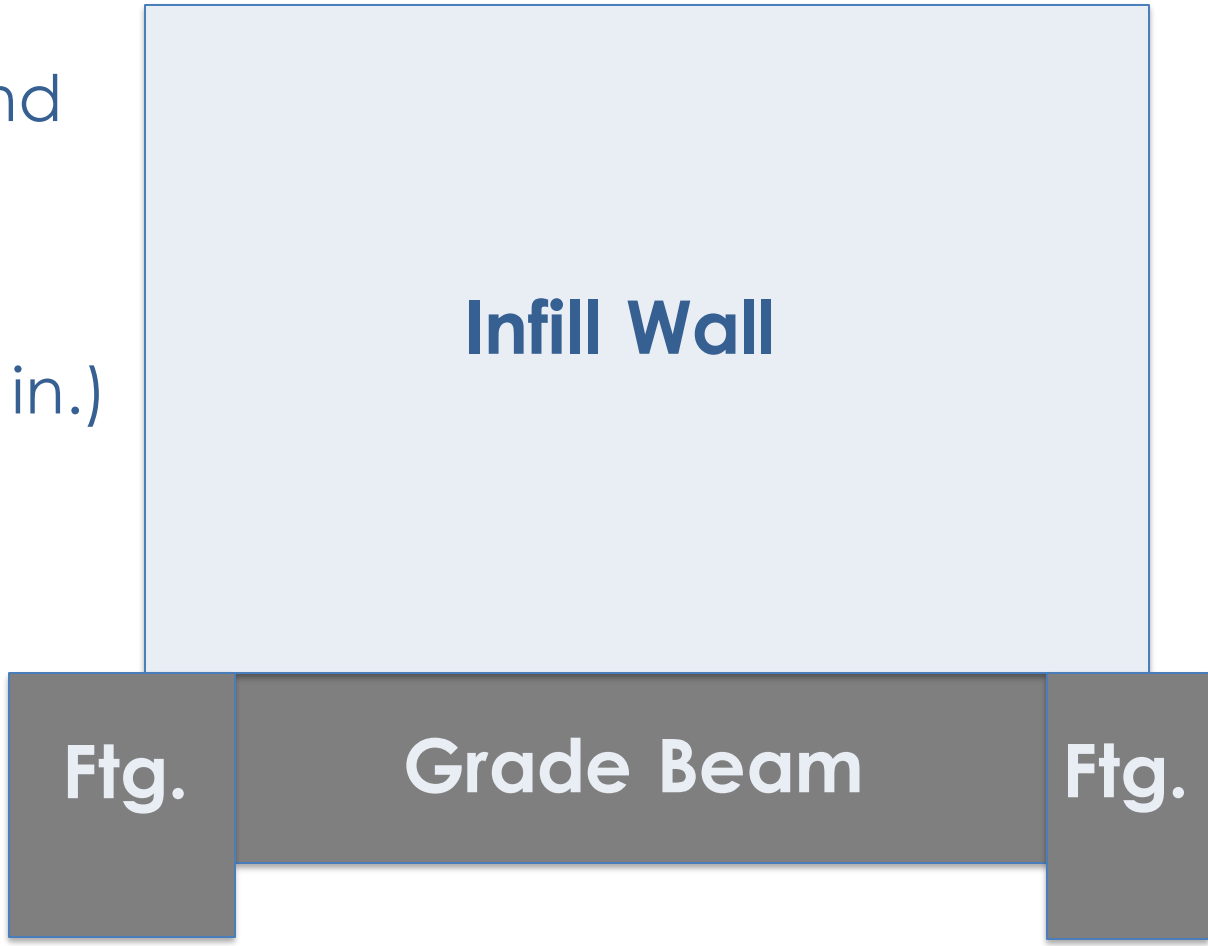
$$\phi V_c = 226 \text{ kip} < 231 \text{ kip} \therefore \text{ Say OK?}$$

Examples: SP-17(14) 11.6 Foundation Example 1

- **Foundation ϕV_c ACI 318-19 < ϕV_c ACI 318-14**
  - 318-19 for this example given is ~1/2 of ACI 318-14
  - Effect of low ρ_w
- **Design impact**
  - Increased thickness; or
  - Increase flexural reinforcement; or
  - Increase concrete strength; or
  - Combination

Examples: Grade beam

- **Infill wall**
  - V_u~1 kip/ft
  - V_u~8.3 kip ea. end
- **Grade beam**
  - b_w =12 in.
  - d = 20 in. (h = 24 in.)
  - f'_c = 4000 psi
  - ℓ = 20 ft
  - ρ_w = 0.0033



Examples: Grade beam

- **Infill wall**
  - V_u~1 kip/ft
  - V_u~8.3 kip ea. end
- **Grade beam**
  - b_w =12 in.
  - d = 20 in. (h = 24 in.)
  - f'_c = 4000 psi
  - ℓ = 20 ft
  - ρ_w = 0.0033

- **ACI 318-14**

$$\phi V_c = \phi 2\lambda\sqrt{f'_c}\,b_w d$$

$$\phi V_c = 0.75(2)(1)\sqrt{4000}(12)(20)$$

$$\phi V_c = 22.8kip \therefore OK$$

$$V_u < (1/2)\phi V_c \therefore A_{v,\min} \text{ not required}$$

- **ACI 318-19**

$$\phi V_c = \phi 8\lambda_s\lambda(\rho_w)^{1/3}\sqrt{f'_c}\,b_w d$$

$$\lambda_s = \sqrt{\frac{2}{1+\frac{20}{10}}} = 0.82$$

$$\phi V_c = 0.75(8)(0.82)(1)(0.0033)^{1/3}\sqrt{4000}(12)(20)$$

$$\phi V_c = 11.1kip \therefore OK$$

$$V_u < \phi\lambda\sqrt{f'_c}\,b_w d = 11.4kip \therefore A_{v,\min} \text{ not required}$$

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Two-way Shear Equations

Why two-way shear provisions changed in 318-19

- **Eqn. developed in 1963 for slabs with t < 5 in. and ρ > 1%**
- **Two issues similar to one-way shear**
  - Size effect
  - Low ρ

Table 22.6.5.2 – Calculation of v_c for two-way shear

v_c		
Least of (a), (b), and (c):	$4\lambda\sqrt{f_c'}$	(a)
	$\left(2+\frac{4}{\beta}\right)\lambda\sqrt{f_c'}$	(b)
	$\left(2+\frac{\alpha_s d}{b_o}\right)\lambda\sqrt{f_c'}$	(c)

Two-way shear size effect

- **Table 22.6.5.2—v_c for two-way members without shear reinforcement**

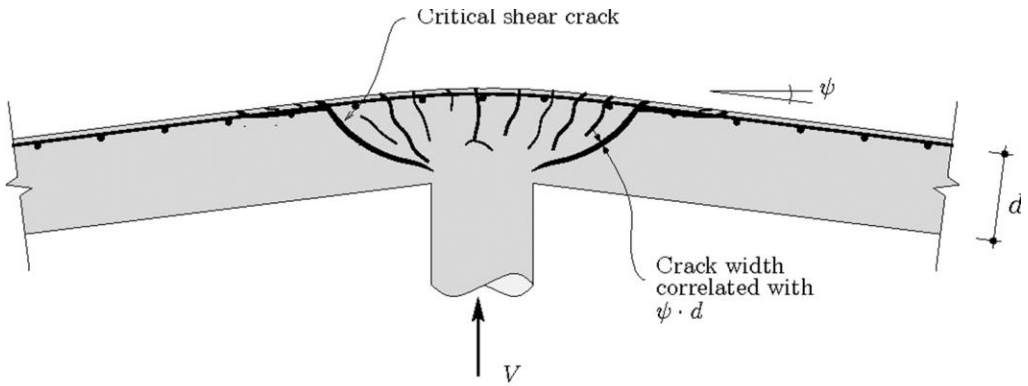
v_c		
Least of (a), (b), and (c):	$4\lambda_s\lambda\sqrt{f_c'}$	(a)
	$\left(2+\frac{4}{\beta}\right)\lambda_s\lambda\sqrt{f_c'}$	(b)
	$\left(2+\frac{\alpha_s d}{b_o}\right)\lambda_s\lambda\sqrt{f_c'}$	(c)

where

$$\lambda_s = \sqrt{\frac{2}{1+\frac{d}{10}}} \le 1$$

Two-way shear low ρ effect

- D, L only, cracking ~$2\sqrt{f'_c}$; punching $4\sqrt{f'_c}$
- Aggregate interlock
- Low ρ ➔ bar yielding, ↑ rotation, ↑crack size, allows sliding of reinforcement
- Punching loads < $4\sqrt{f'_c}$



Why two-way shear provisions changed in 318-19: New two-way slab reinforcement limits 8.6.1—Reinforcement limits

- $A_{s,min} \geq 0.0018 A_g$
- If $v_{uv} > \phi 2\lambda_s \lambda \sqrt{f_c'}$ on the critical section
- Then $A_{s,\min} \geq \dfrac{5 v_{uv} b_{slab} b_o}{\phi \alpha_s f_y}$

Why two-way shear provisions changed in 318-19: 8.4.2.2.3

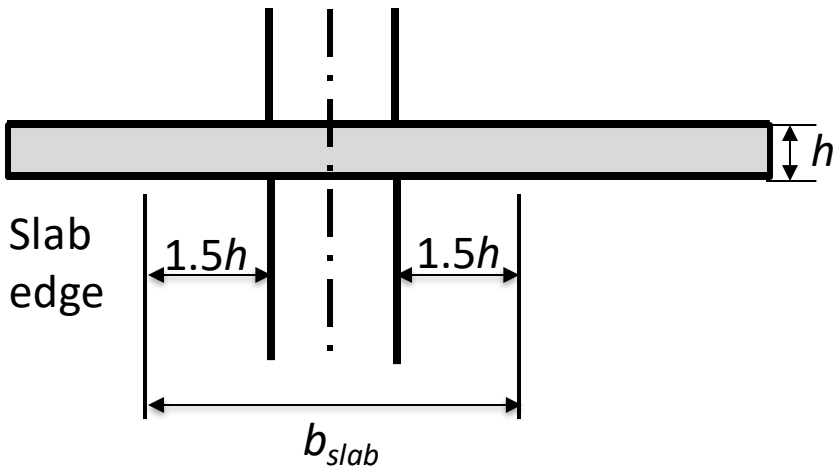
8.4.2.2.3 The effective slab width b_{slab} for resisting $\gamma_f M_{sc}$ shall be the width of column or capital plus a distance on each side in accordance with Table 8.4.2.2.3.

Table 8.4.2.2.3—Dimensional limits for effective slab width

	Distance on each side of column or capital	
Without drop panel or shear cap	Lesser	1.5h of slab
		Distance to edge of slab
With drop panel or shear cap	Lesser	1.5h of drop or cap
		Distance to edge of the drop or cap plus 1.5h of slab

Table 8.4.2.2.3

b_{slab} is the lesser of:



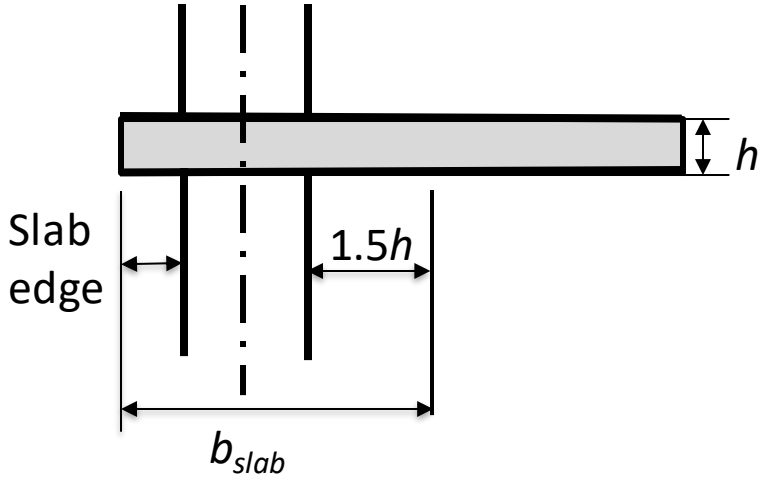
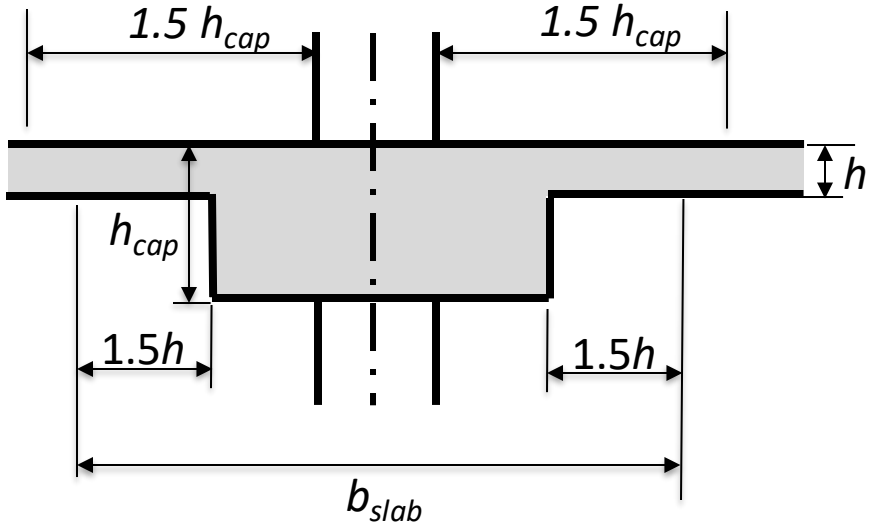
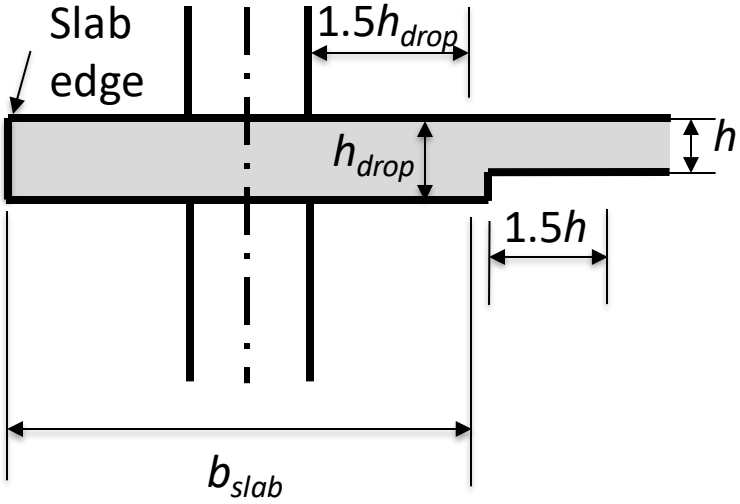


Table 8.4.2.2.3

b_{slab} is the lesser of:





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Wall Shear Equations



Coordination of Chap. 11 and 18 Wall Shear Eqs.

- **ACI 318-83 introduced seismic equation**
  - Two wall shear equation forms
- **Equation forms gave similar results**
- **Committee 318 wanted consistency in form**

Coordination of Chap. 11 and 18 Wall Shear Eqs.

- Chapter 11: all changes
- Chapter 18: no change
- **318-14 simplified compression eq. (Table 11.5.4.6)**

$$V_n = 2\lambda\sqrt{f_c'}hd + \frac{A_v f_{yt} d}{s}$$

Coordination of Chap. 11 and 18 Wall Shear Eqs.

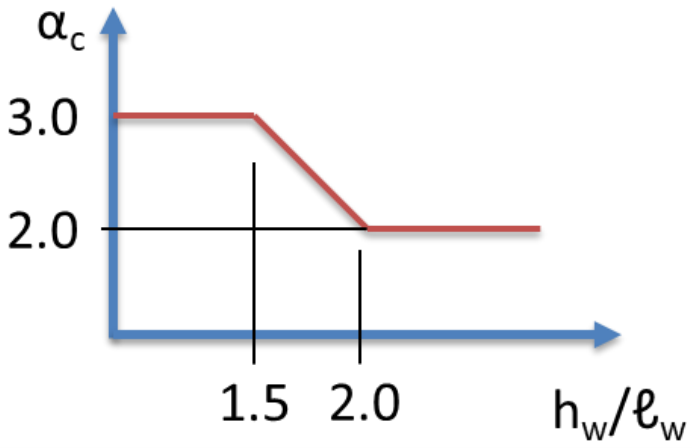
- **318-19 Eq. 11.5.4.3**

$$V_n = \left(\alpha_c \lambda \sqrt{f_c'} + \rho_t f_{yt} \right) A_{cv}$$

- **318-19 Eq. 18.10.4.1 (same as -14)**

$$V_n = \left(\alpha_c \lambda \sqrt{f_c'} + \rho_t f_{yt} \right) A_{cv}$$

- α_c



Coordination of Chap. 11 and 18 Wall Shear Eqs.

- **Impact minor**
- **Similar results 318-14 to 19**
- **Note use of ℓ_w in 318-19 vs d in 318-14**
  - d in 318-14 assumed **0.8 ℓ_w**
  - Results in a "lower" max V_n:

$$V_n = 10\sqrt{f'_c}hd \quad (318-14)$$

$$V_n = 8\sqrt{f'_c}h\ell_w \quad (318-19)$$
$$= 8\sqrt{f'_c}A_{cv}$$

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Spacing of Shear Reinforcement

Maximum spacing of legs of shear reinforcement

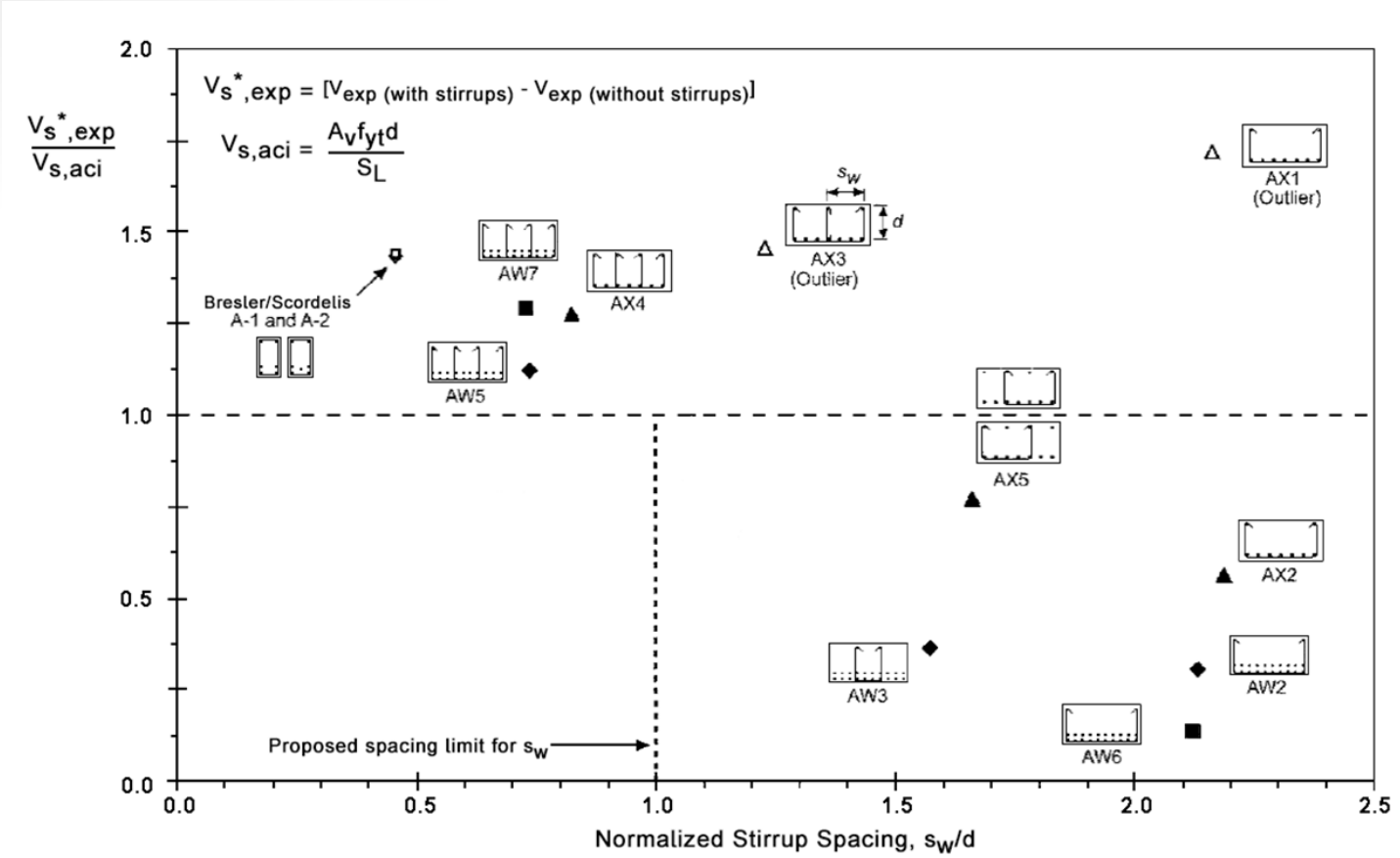


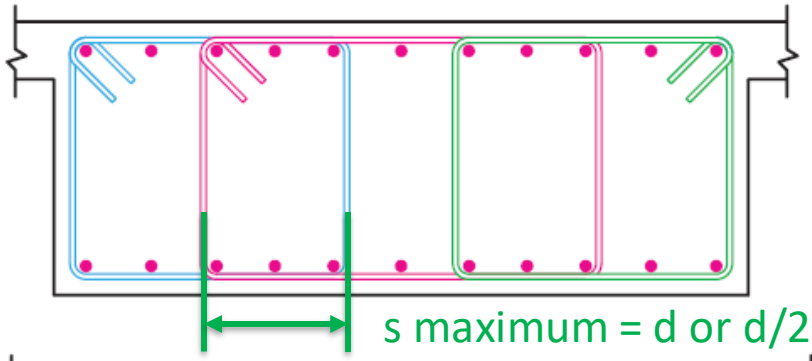
Fig. 10—Influence of stirrup leg spacing across width on effectiveness of shear reinforcement.

Source: Lubell et. al, "Shear Reinforcement Spacing in Wide Members, ACI Structural Journal 2009

Table 9.7.6.2.2—Maximum spacing of legs of shear reinforcement

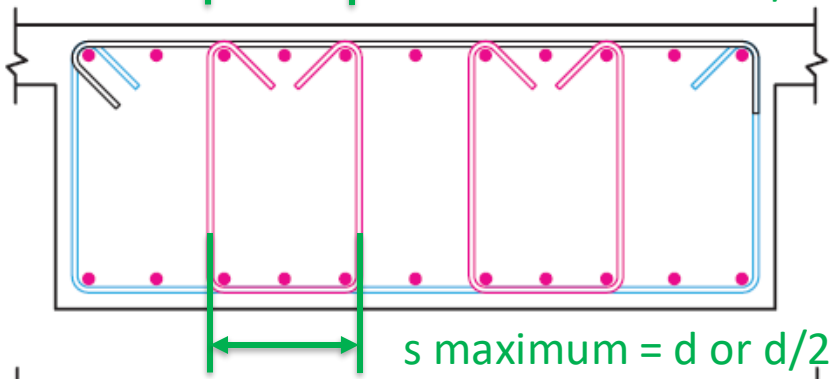
Required V_s		Maximum s, in.			
		Nonprestressed beam		Prestressed beam	
		Along length	Across width	Along length	Across width
$\leq 4\sqrt{f_c'}b_w d$	Lesser of:	$d/2$	d	$3h/4$	$3h/2$
		24 in.			
$> 4\sqrt{f_c'}b_w d$	Lesser of	$d/4$	$d/2$	$3h/8$	$3h/4$
		12 in.			

Maximum spacing of legs of shear reinforcement



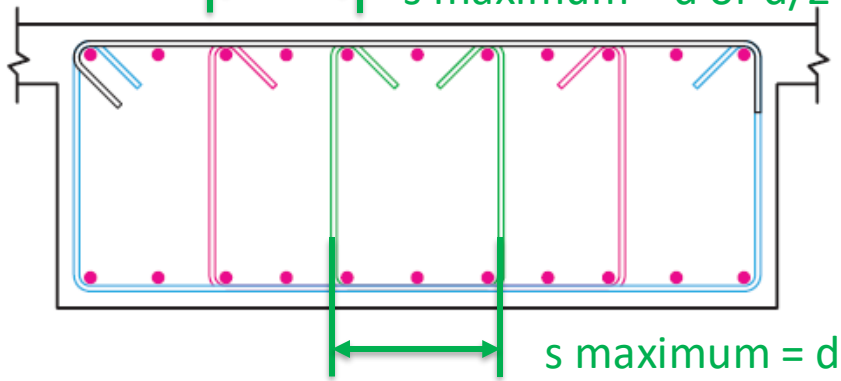
Beam stirrup configuration with three closed stirrups distributed across the beam width

s maximum = d or d/2 nonprestressed, 3h/2 or 3h/4 prestressed



Single U-stirrup (with 135-degree hooks) across the net width of the beam, two identical U-stirrups (each with 135-degree hooks) distributed across the beam interior, and a stirrup cap

s maximum = d or d/2 nonprestressed, 3h/2 or 3h/4 prestressed



Single U-stirrup across the net width of the beam, two smaller-width U-stirrups nested in the beam interior, and a stirrup cap

s maximum = d or d/2 nonprestressed, 3h/2 or 3h/4 prestressed

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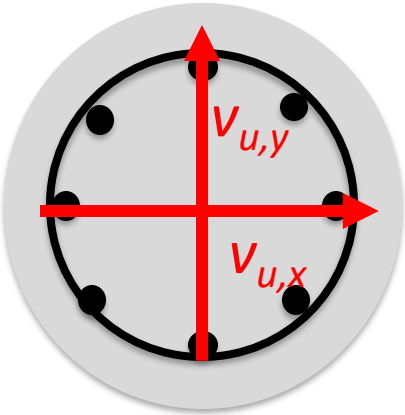


Bi-directional Shear

Interaction of shear forces

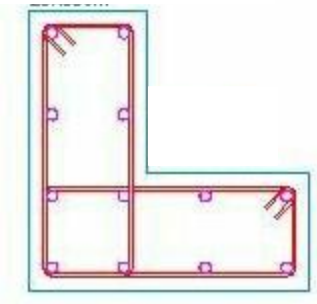
- **Biaxial shear**
- **Symmetrical RC circular sections**
  - ϕV_c equal about any axis
  - V_u on 2 centroidal axes, V_u = resultant

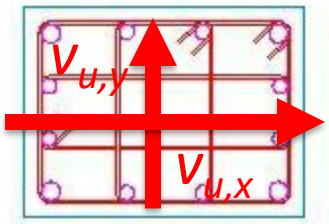
$$v_u = \sqrt{(v_{u,x})^2 + (v_{u,y})^2}$$

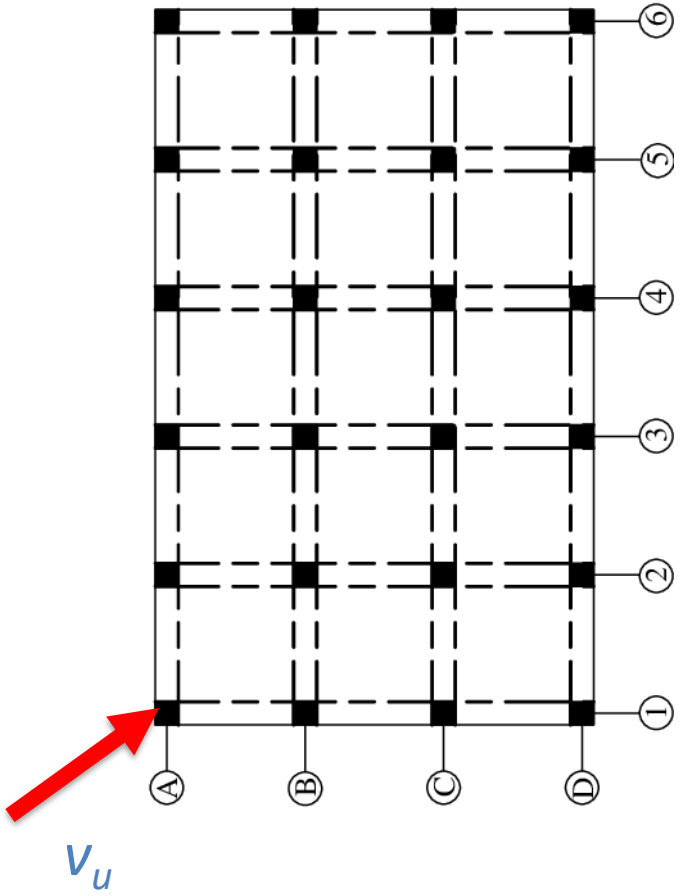


Interaction of shear forces

- **Biaxial shear**
- **Rectangular RC sections**
  - ϕV_c differs between axes
  - V_u on 2 axes, $\phi V_c \neq$ resultant

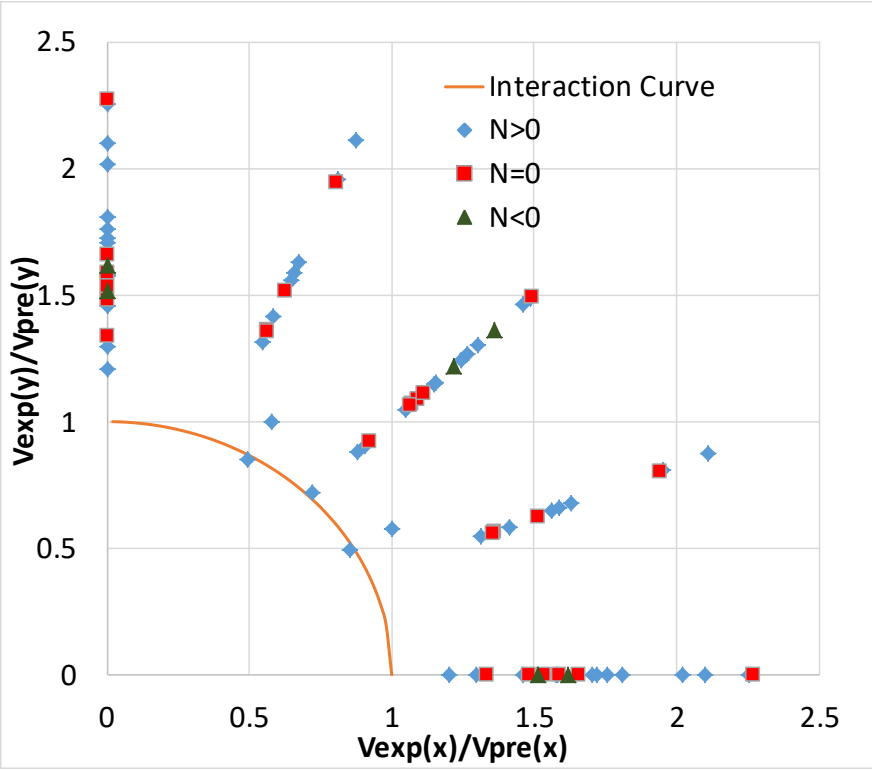






Interaction of shear forces

- Biaxial shear on non-circular cross section
- ϕV_c = Elliptical interaction diagram



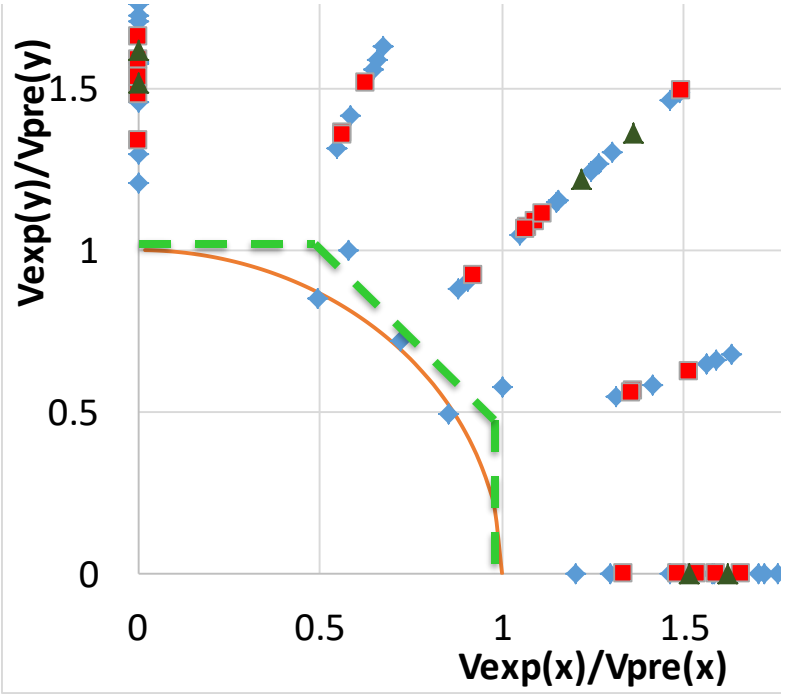
Interaction of shear forces

- **22.5.1.10 Neglect interaction of shear forces**

  If $v_{u,x}/\phi v_{n,x} \leq 0.5$, or $v_{u,y}/\phi v_{n,y} \leq 0.5$

- **22.5.1.11 requires interaction consideration**

  If $v_{u,x}/\phi v_{n,x} > 0.5$, and $v_{u,y}/\phi v_{n,y} > 0.5$, then



$$\frac{v_{u,x}}{\phi v_{n,x}} + \frac{v_{u,y}}{\phi v_{n,y}} \leq 1.5$$

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Hanger Reinforcement

Monolithic beam-to-beam joints: Hanger steel

- **Commentary added: R9.7.6.2**
- **Hanger reinforcement**
  - Suggested where both the following are true:
  - Beam depth ≥ 0.5 girder depth
  - Stress transmitted from beam to girder ≥ $3\sqrt{f'_c}$ of the beam

Monolithic beam-to-beam joints: Hanger steel

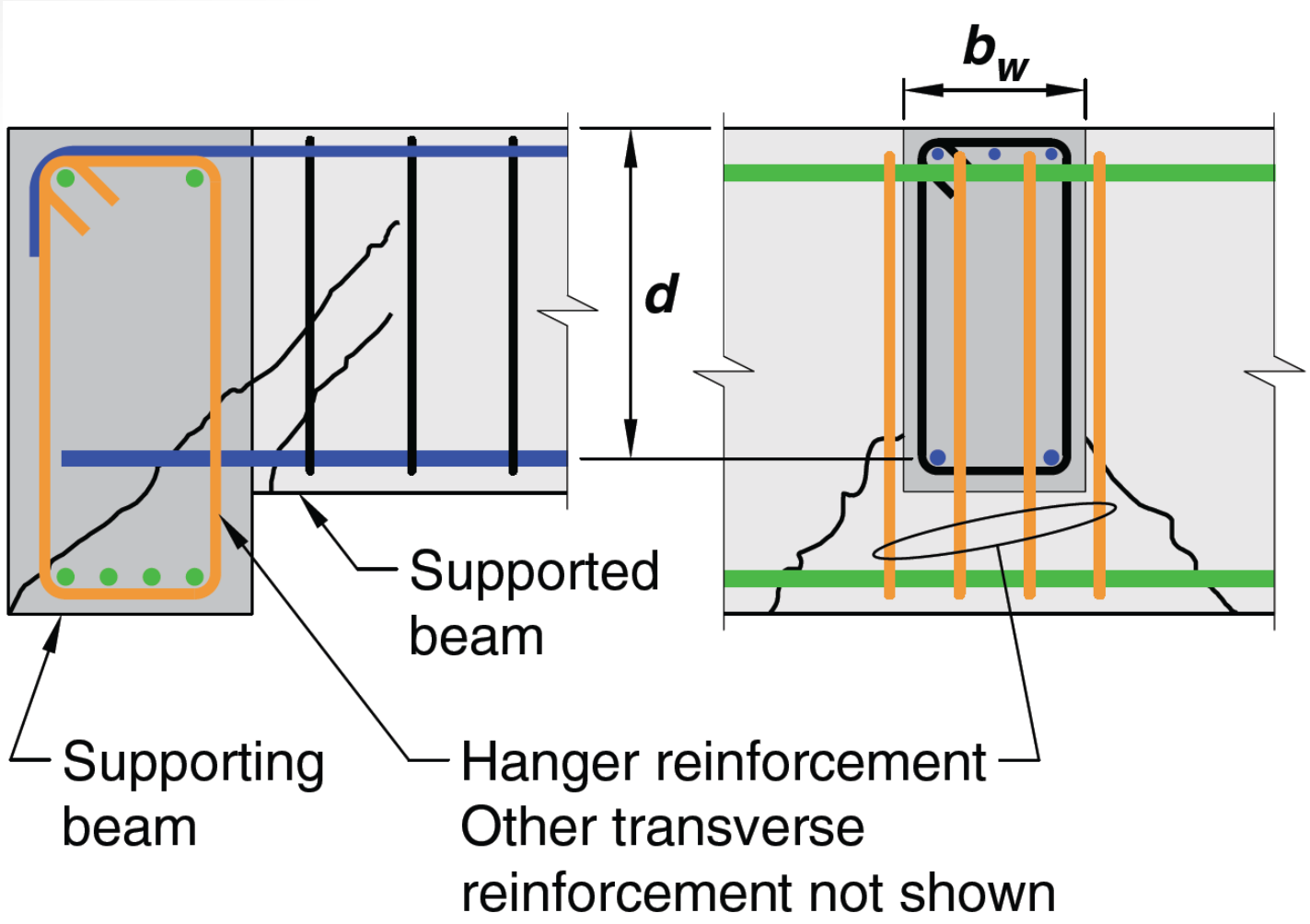


Fig. R9.7.6.2.1—Hanger reinforcement for shear transfer.

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Concrete Durability and Materials

Changes in durability and materials

- **Changes in material properties (19.2)**
  - Additional minimum f'_c requirements
  - E_c requirements
- **Changes in durability (19.3)**
  - Calculating chloride ion content
  - Sulfate exposure class **S3**
  - Water exposure class **W**
  - Corrosion exposure class **C0**
- **Changes in material (26.4.1)**
  - Alternative cements
  - New aggregates
    - Recycled aggregates
    - Mineral fillers
- **Evaluation and acceptance (26.12)**
  - Strength tests
- **Inspection (26.13)**

Table 19.2.1.1 – Additional minimum strength, f'_c

Structural walls in SDC D, E, and F	Min. f'_c (psi)
Special structural walls with Grade 100 reinforcement	5000

Higher strength concrete used with higher strength steel

- Enhances bar anchorage
- Reduces neutral axis depth for improved performance

19.2.2.1R Modulus of Elasticity

- E_c from Code equations is appropriate for most applications

- Large differences for HSC (f'_c > 8000 psi), LWC, and mixtures with low coarse of aggregate volume

19.2.2.2 Modulus of Elasticity

E_c can be specified based on testing of concrete mixtures:

a) Use of specified E_C for proportioning concrete mixture

b) Test for specified E_C

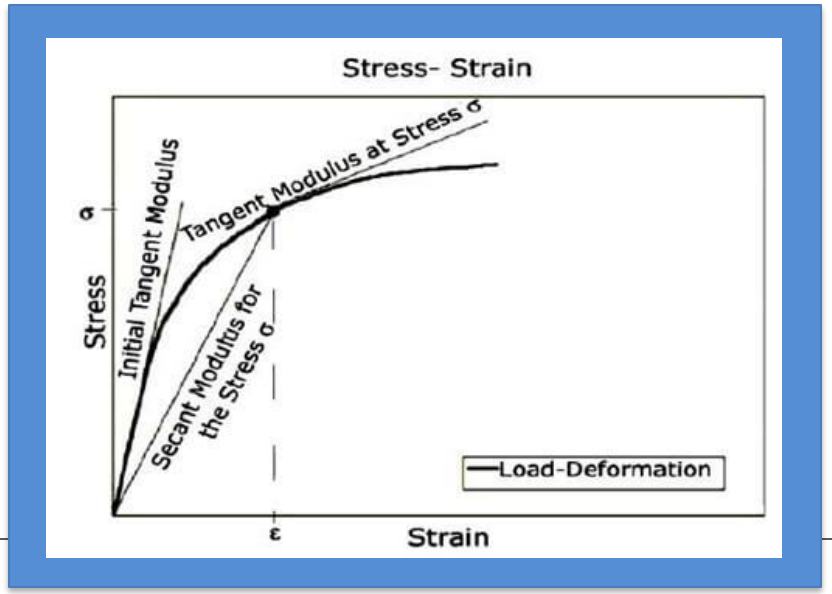
c) Test for E_C at 28 days or as indicated in construction documents



Source: Engineering discoveries 346

Contract Document Information

- **Members for which E_c testing of concrete mixtures is required (26.3.1(c))**
- **Proportioning (26.4.3.1(c))**
  - E_c is average of 3 cylinders
  - Cylinders made and cured in the lab
  - $E_c \geq$ specified value



Source: Engineering Discoveries



Changes in durability and materials

- **Changes in material properties (19.2)**
  - Additional minimum f'_c requirements
  - *Ec* requirements
- **Changes in durability (19.3)**
  - Calculating chloride ion content
  - Sulfate exposure class **S3**
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  - Strength tests
- **Inspection (26.13)**

Table 19.3.2.1 – Allowable chloride limits

- Percent mass of total cementitious materials rather than percent weight of cement

Class	Max w/cm	Min. f'_c, psi	Maximum water-soluble chloride ion (Cl^-) content in concrete, by percent mass of cementitious materials		Additional provisions
			Non-prestressed concrete	Prestressed concrete	
C0	N/A	2500	1.00	0.06	None
C1	N/A	2500	0.30	0.06	
C2	0.40	5000	0.15	0.06	Cover per 20.5

For calculation, cementitious materials ≤ cement

Determining chloride ion content

- **26.4.2.2(e) - 2 methods to calculate total chloride ion content**

(1) Calculated from chloride ion content from concrete materials and concrete mixture proportions

(2) Measured on hardened concrete in accordance with ASTM C1218 at age between 28 and 42 days

Sulfate Attack – Change in S3



Credit: PCA

Table 19.3.2.1 – Exposure Category S – 'S3' Options 1 and 2

Class	Max. w/cm	Min. f'c (psi)	Cementitious Materials, Type			Calcium chloride admixture
			C150	C595	C1157	
SO	N/A	2500				No restriction
S1	0.50	4000	II	~~IP, IS, or IT~~ Types with (MS)	MS	No restriction
S2	0.45	4500	V	~~IP, IS, or IT~~ Types with (HS)	HS	Not permitted
S3 Option 1	0.45	4500	V + Pozz or slag	~~IP, IS, or IT~~ Types with (HS) + Pozz or slag	HS + Pozz or Slag	Not permitted
S3 Option 2	0.40	5000	V	Types with (HS)	HS	Not permitted

Added advantage of sulfate exposure S3 – Option 2

- Option 1: 18 month test results
- Option 2: 6 and 12 month test results

Table 26.4.2.2(c)—Requirements for establishing suitability of combinations of cementitious materials for Exposure Class S

Exposure class		Maximum length change for tests in accordance with ASTM C1012, percent		
		At 6 months	At 12 months	At 18 months
S1		0.10	No requirement	No requirement
S2		0.05	0.10[1]	No requirement
S3	Option 1	No requirement	No requirement	0.10
	Option 2	0.05	0.10[1]	No requirement



Table 19.3.2.1 – Water Exposure Category W

Two Categories – concrete in contact with water: W1 and W2

Class	Condition	Example
WO	Concrete dry in service	Interior concrete
W1	**Concrete in contact with water where low permeability is not required**	**Foundation member below water table**
W2	Concrete in contact with water where low permeability is required	Pavement parking deck surface

Class	Max. w/cm	Min. f'c (psi)	Additional requirements
WO	N/A	2500	none
W1	**N/A**	**2500**	**26.4.2.2(d)**
W2	0.50	5000	26.4.2.2(d)

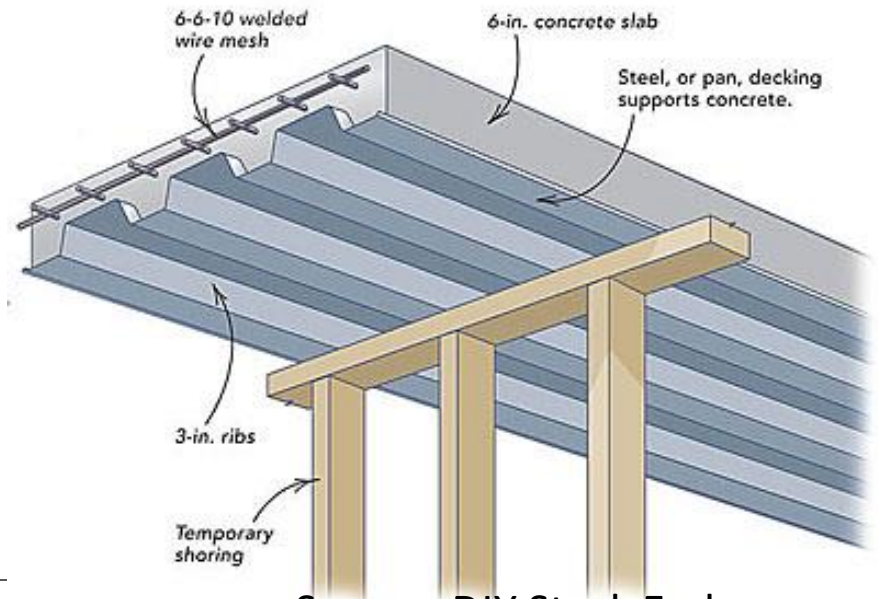
Exposure W1 and W2 check for reactive aggregates

- **26.4.2.2(d)** – Concrete exposed to W1 and W2, concrete mixture to comply with
  - ASR susceptible aggregates not permitted unless mitigated
  - ACR susceptible aggregates not permitted



26.4.2 Concrete Mixture Requirements

26.4.2.2(g) Concrete placed on or against stay-in-place **galvanized steel forms**, **max. chloride ion content** shall be **0.30 percent** by mass of cementitious materials unless a more stringent limit for the member is specified



Source: DIY Stack Exchange

Changes in durability and materials

- **Changes in material properties (19.2)**
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  - *Ec* requirements
- **Changes in durability (19.3)**
  - Calculating chloride ion content
  - Sulfate exposure class **S3**
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- **Changes in material (26.4.1)**
  - Alternative cements
  - New aggregates
    - Recycled aggregates
    - Mineral fillers
- **Evaluation and acceptance (26.12)**
  - Strength tests
- **Inspection (26.13)**

New materials allowed



- **Alternative cements (26.4.1.1)**
  - Inorganic cements used as 100% replacement of PC
  - Recycled glass and others in ITG-10

- **Alternative aggregates and mineral fillers (26.4.1.2 and 3)**
  - Recycled aggregated from crushed concrete
  - Mineral fillers – finely ground recycled glass or others

New materials allowed



Permitted if:

- Documented test data confirms mechanical properties are met for design of structural concrete (strength, durability, fire)
- Approved by LDP and Building official
- Ongoing testing program and QC program (alternative recycled aggregates) to achieve consistency of properties of concrete

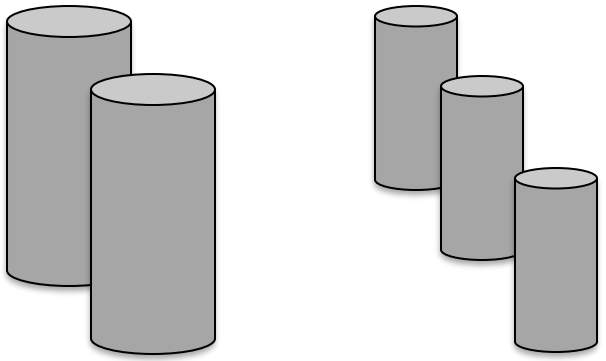


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  - New aggregates
    - Recycled aggregates
    - Mineral fillers
- **Evaluation and acceptance (26.12)**
  - Strength tests
- **Inspection (26.13)**

26.12—Evaluation and acceptance of hardened concrete

- **26.12.1.1**
  - Added ASTMs for sampling, cylinders, and testing
  - Sample taken at point of delivery
  - Certified field and lab testing technicians required
  - Clarified that "Strength test" is the average of at least two 6 x 12 in. or three 4 x 8 in. cylinders



26.12.6 Investigation of strength tests

(d) Cores testing:

- Min. 5 days after being wetted
- Max. 7 days after coring

Unless otherwise approved by LDP or building official



Source: The Constructor



Changes in durability and materials

- **Changes in material properties (19.2)**
  - Additional minimum f'_c requirements
  - *Ec* requirements
- **Changes in durability (19.3)**
  - Calculating chloride ion content
  - Sulfate exposure class **S3**
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  - Alternative cements
  - New aggregates
    - Recycled aggregates
    - Mineral fillers
- **Evaluation and acceptance (26.12)**
  - Strength tests
- **Inspection (26.13)**

26.13—Inspection

26.13.1.1 Concrete construction inspection per:

- General building code (GBC)
- ACI 318 in absence of GBC



Source: Galvanizeit

26.13—Inspection

Inspector must be certified when inspecting:

- Formwork,
- Concrete placement,
- Reinforcement,
- Embedments



Photo courtesy Larry Novak

Seismic Inspections (26.13.1.3)

Inspection performed by:

- LDP responsible for the design
- An individual under the supervision of LDP
- Certified inspector

Elements to be inspected:

- Placement and reinforcement for SMF
- Boundary elements of SSW,
- Coupling beams, and
- Precast concrete diaphragms in SDC C, D, E, or F using moderate or high-deformability connections
- Tolerances of precast concrete diaphragm connections per ACI 550.5



Source: NIST page



Other Inspections (26.13.1)

- Reinforcement welding → qualified welding inspector
- Expansion, screw, and undercut anchors → inspector certified or approved by LDP and building official
- Adhesive anchors → certified inspector

26.13.3.2 Items requiring continuous inspection

26.13.3.2 Items requiring verification and continuous inspection shall include (a) through (e):

(a) Prior to placement, concrete mixture for intended location.

(b) Tensioning of prestressing reinforcement and grouting of bonded tendons.

(c) Placement of reinforcement for special moment frames, boundary elements of special structural walls, and coupling beams.

(d) Welding of reinforcement for special moment frames, boundary elements of special structural walls, and coupling beams.

(e) Post-installed anchor installation, if required as a condition of the anchor assessment or if adhesive anchors are installed in horizontal or upwardly inclined orientations to resist sustained tensile loads.

26.13.3.3 Items requiring periodic inspection

26.13.3.3 Items requiring verification and periodic inspection shall include (a) through (j):

(a) Placement of reinforcement, embedments, and post-tensioning tendons.
(b) Welding of reinforcement except as required in 26.13.3.2(d).
(c) Curing method and duration of curing for each member.
(d) Construction and removal of forms and reshoring.
(e) Sequence of erection and connection of precast members.

26.13.3.3 Items requiring periodic inspection

(f) Verification of in-place strength of concrete before stressing post-tensioned tendons and before removal of shores and formwork from beams and structural slabs.

(g) Placement of cast-in-anchors and anchor reinforcement, including tolerances required for location of anchor reinforcement.

(h) Installation of post-installed expansion (torque-controlled and displacement-controlled) screw, and undercut anchors.

(i) Installation of adhesive anchors, except as required in 26.13.3.2(e).

(j) Proof loading of anchors if required in accordance with 26.13.2.5.

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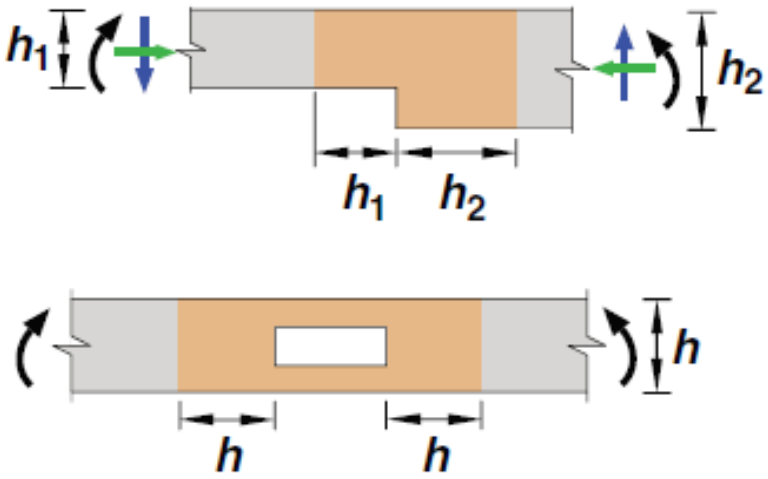
Changes to the Concrete Design Standard

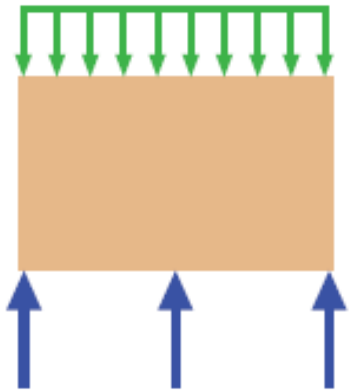


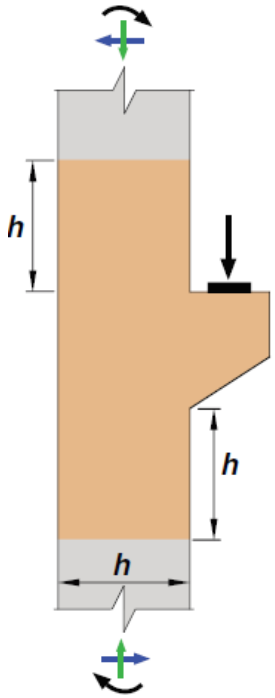
Strut-and-Tie Method

Why strut-and-tie method?

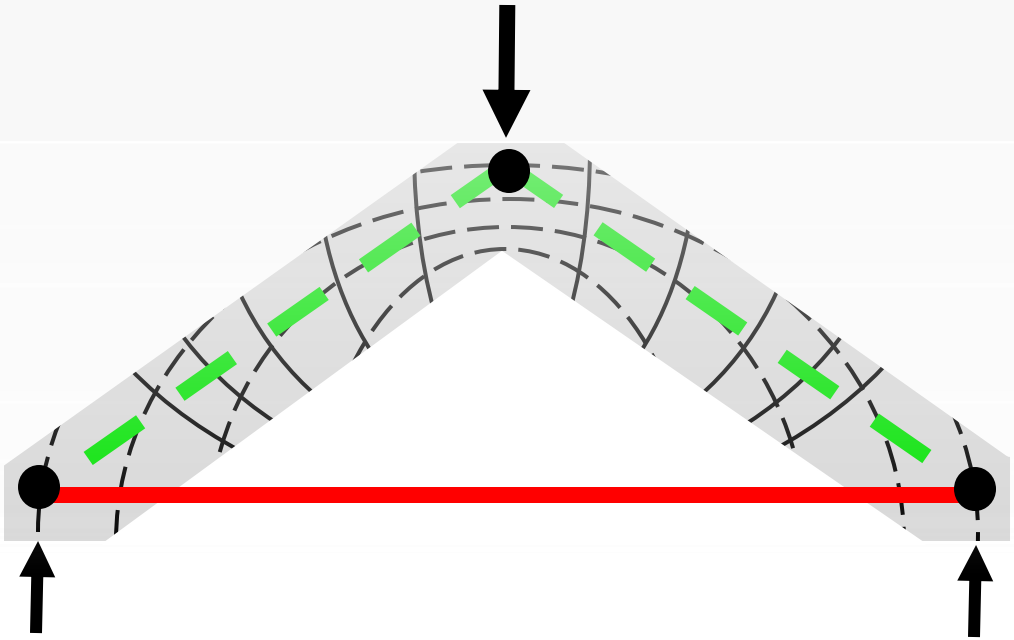
- Valuable tool where plane-sections assumption of beam theory does not apply
- Truss analogy used to analyze concrete structures







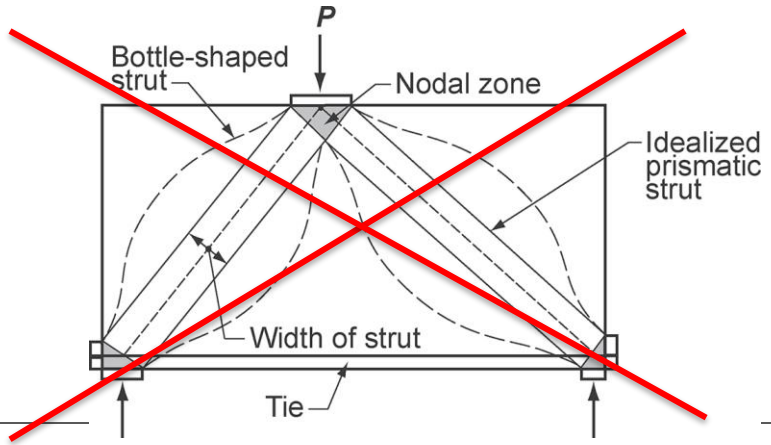
Strut and Tie Method

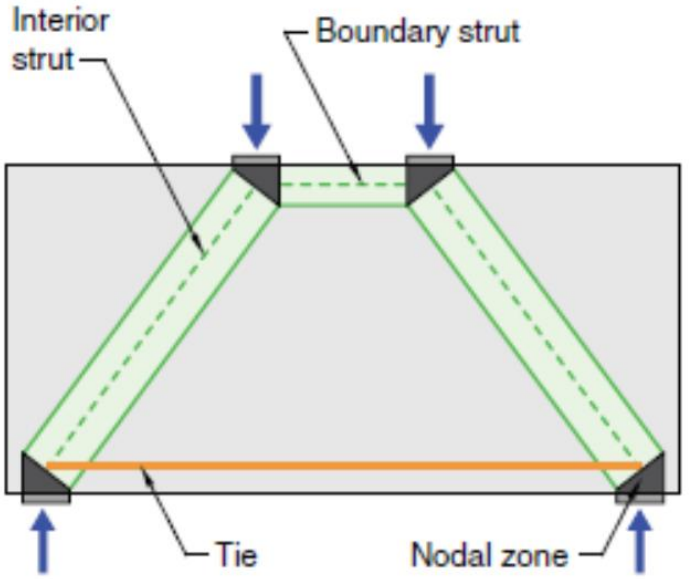


Deletion of bottle-shaped strut

Bottle-shaped strut

- Spreads out at a slope of 2:1
- Reinforcement is at an angle orthogonal to grid (Not used)
- Requirement deleted







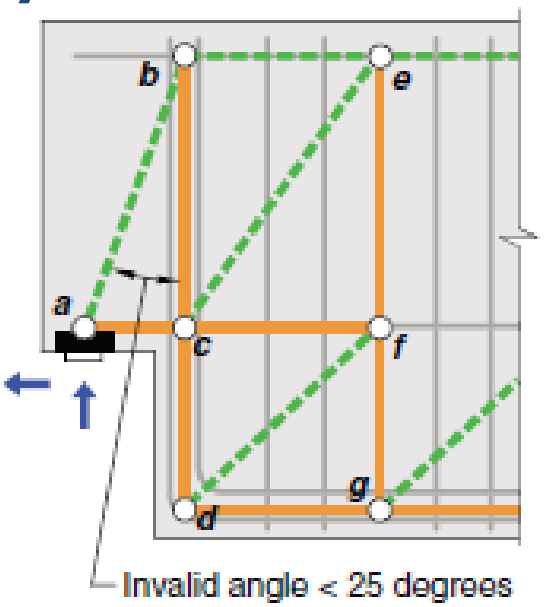
Code Changes—Strut-and-tie method

- Minimum angle between strut and tie
- Effect of prestressing
- Development of tie forces
- Strut strength and maximum shear stress
- Minimum reinforcement in D-region
- Curved nodes
- STM part of seismic force resisting system

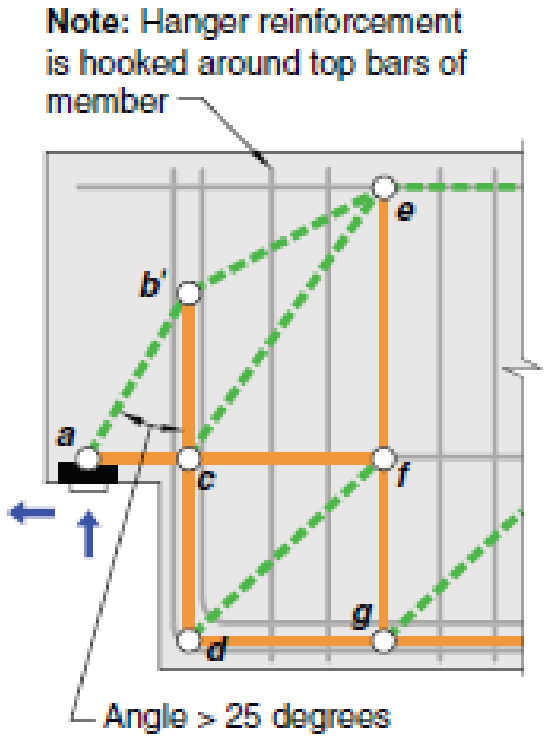
R23.2.7 Angle between strut and tie

$25° \leq \theta \leq 65°$

- Mitigate cracking
- Compatibility



(a) Invalid strut-and-tie model



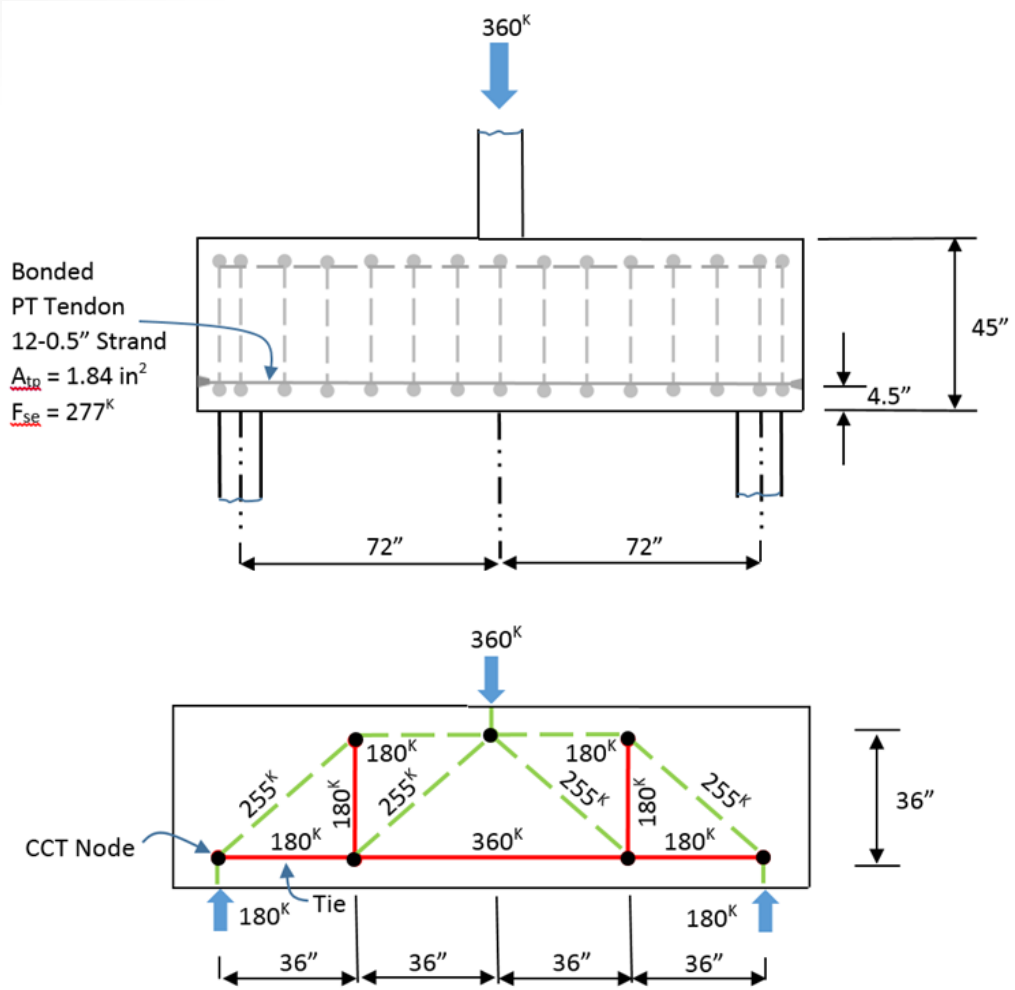
(b) Adjusted strut-and-tie model to satisfy 23.2.7



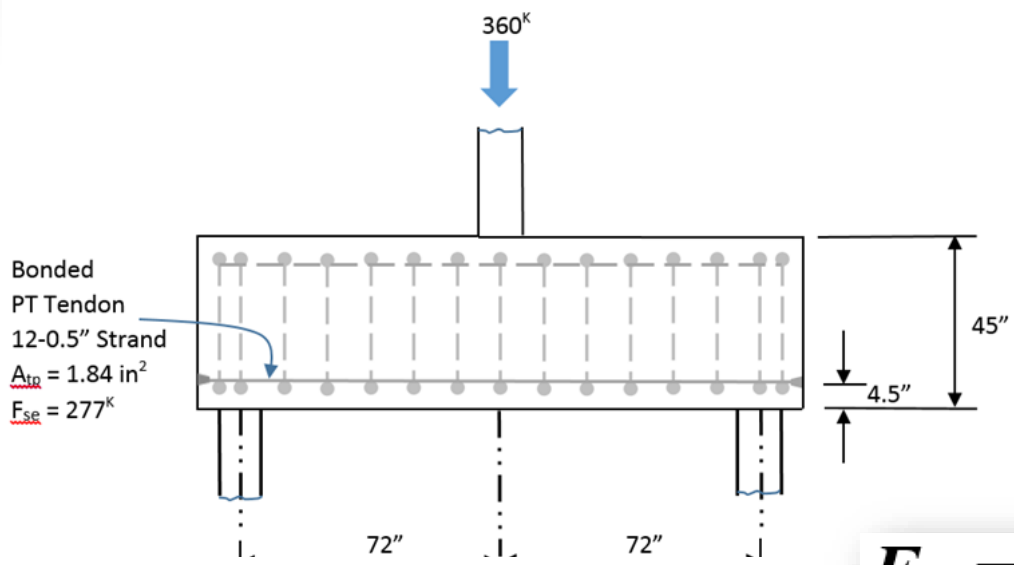
Code Changes—Strut-and-tie method

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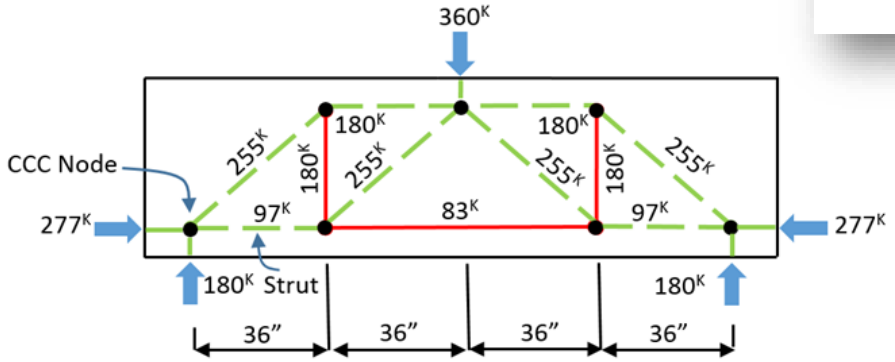
23.2.8 Effect of Prestressing



23.2.8 Effect of Prestressing



$$F_{nt} = A_{ts} f_y + A_{tp} \Delta f_p$$





23.2.8 Effect of Prestressing in STM

- **Use as an external load**
- **Prestress force applied at end of strand transfer length**
- **Load factors per 5.3.13**
  - LF of 1.2 if PT effects increase net force in struts or ties
  - LF of 0.9 if PT reduce net force in struts or ties

23.7 Strength of ties

Tensile strength:

- Simple tension element
- $F_{nt} = A_{ts} f_y + A_{tp} \Delta f_p$
- $\phi = 0.75$ for all ties

- $A_{tp} = 0$ (nonprestressed)
- Δf_p = 60 ksi for bonded prestressed reinf. and
  10 ksi for unbonded prestressed reinf.

$$\Delta f_{p,max} = f_{py} - f_{se}$$

Note: tie centroid coincides with reinforcement centroid



Code Changes—Strut-and-tie method

- Minimum angle between strut and tie
- Effect of prestressing
- **Development of tie forces**
- Strut strength and maximum shear stress
- Minimum reinforcement in D-region
- Curved nodes
- STM part of seismic force resisting system

23.8.2 Strength of ties

Anchorage of tie reinforcement is accomplished by:

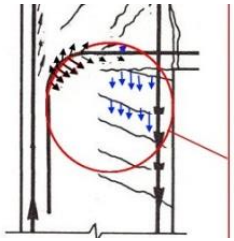
- Mechanical devices



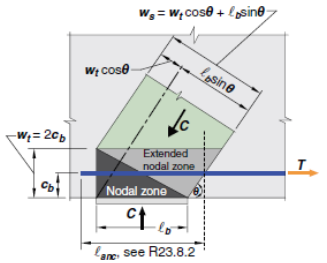
- Post-tensioning anchorage devices



- Standard hooks

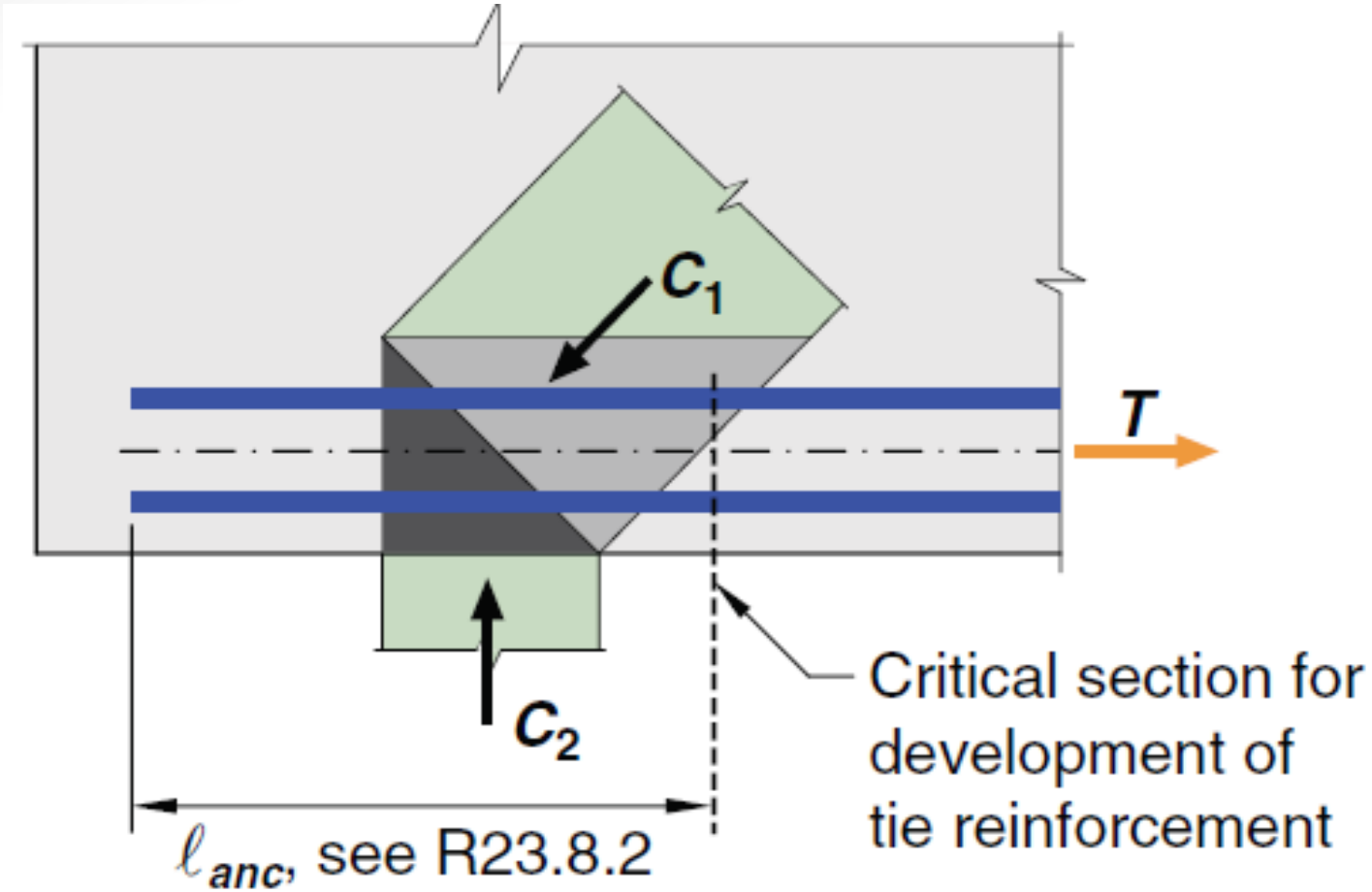


- Straight bar development

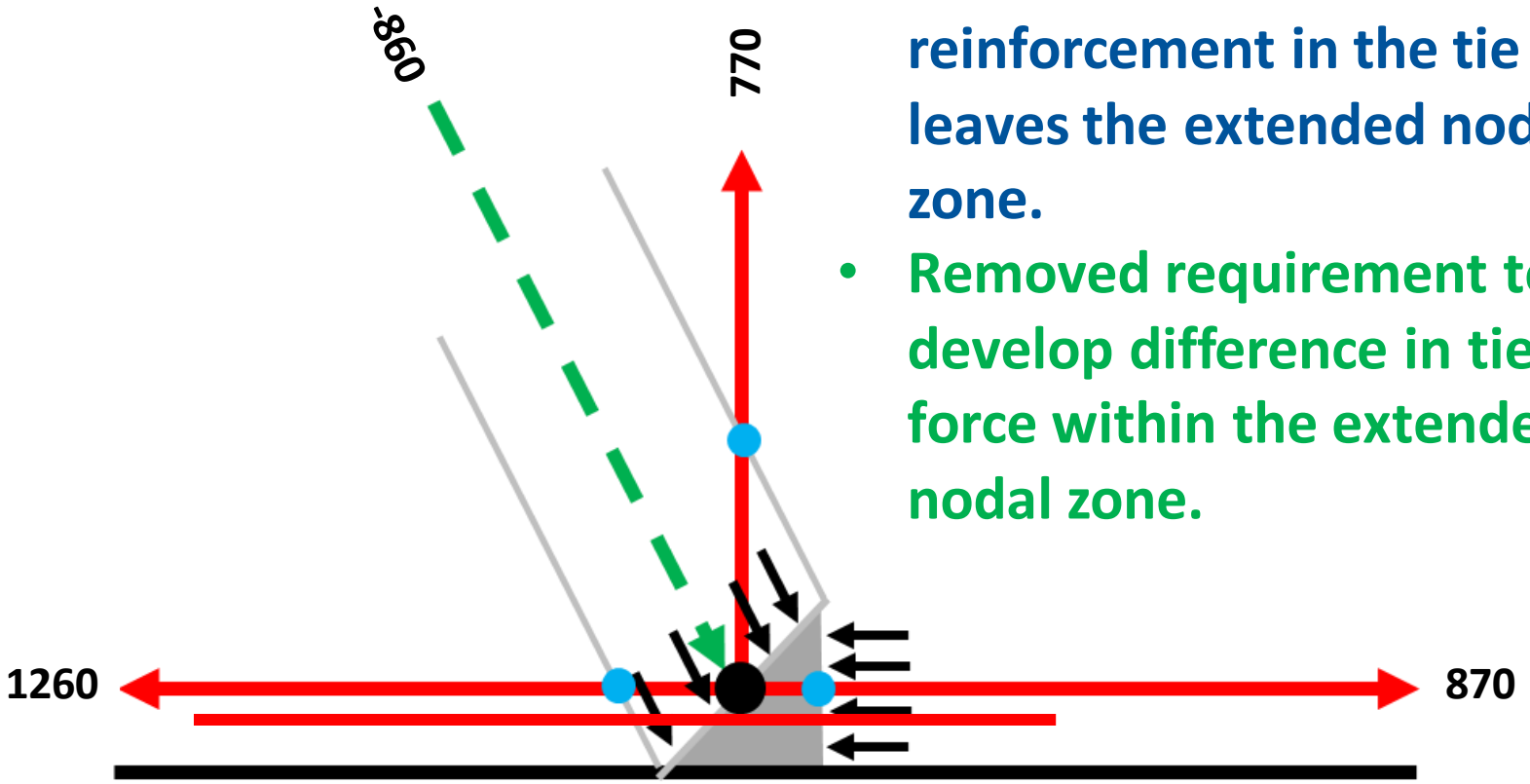


- ***Except*** *ties extending from* ***curved-bar nodes***

23.8.2 Strength of ties



23.8.3 Development of Tie Forces



- Tie force is developed in each direction at the point where the centroid of the reinforcement in the tie leaves the extended nodal zone.
- Removed requirement to develop difference in tie force within the extended nodal zone.

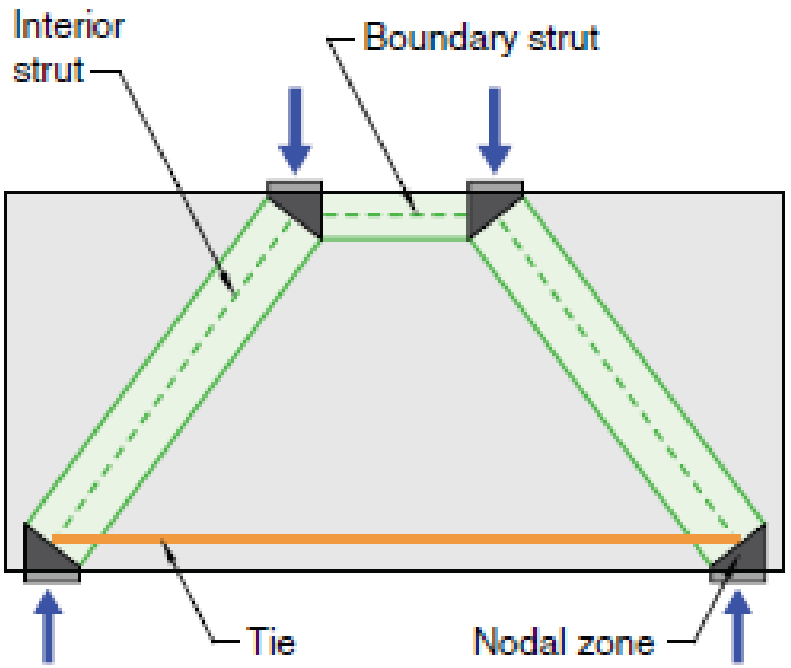


Code Changes—Strut-and-tie method

- Minimum angle between strut and tie
- Effect of prestressing
- Development of tie forces
- **Strut strength and maximum shear stress**
- Minimum reinforcement in D-region
- Curved nodes
- STM part of seismic force resisting system

23.4 Strength of struts

- **3 components**
  - Struts
  - Ties
  - Nodal zones



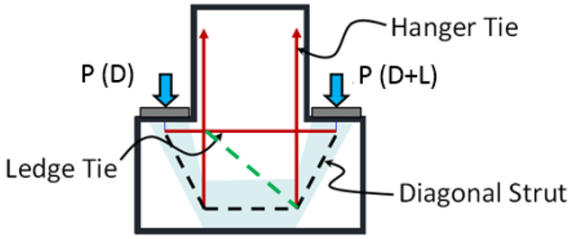
Strut strength:

$$F_{ns} = f_{ce} A_{cs} + A'_s f'_s$$

and

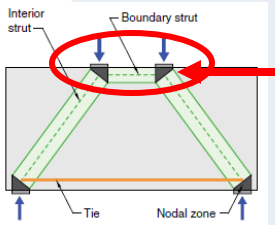
$$f_{ce} = 0.85\beta_c\beta_s f'_c$$

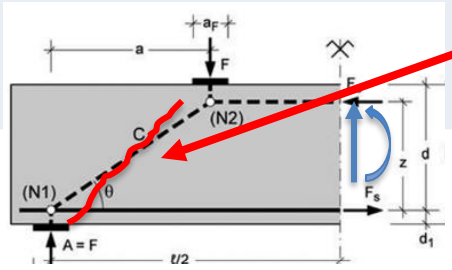
23.4 Strength of struts



Strut coefficient, β_s → Table 23.4.3

Strut location	Strut type	Criteria	β_s	
Tension members or tension zones of members	Any	All cases	0.4	(a)
All other cases	**Boundary strut**	All cases	1.0	(b)
All other cases	**Interior struts**	**Reinforcement satisfying (a) or (b) of Table 23.5.1**	**0.75**	(c)
All other cases	**Interior struts**	$V_u \le \phi 5\lambda\lambda_s\sqrt{f'_c}b_w d\tan\theta$	**0.75**	(d)
All other cases	**Interior struts**	Beam-column joints	0.75	(e)
All other cases	**Interior struts**	All other cases	**0.4**	(f)





23.4 Strength of struts

$$V_u \leq \phi 5 \lambda \lambda_s \sqrt{f'_c}\, b_w d \tan\theta$$

With λ_s:

1- $\lambda_s = 1$ if distributed reinforcement is provided

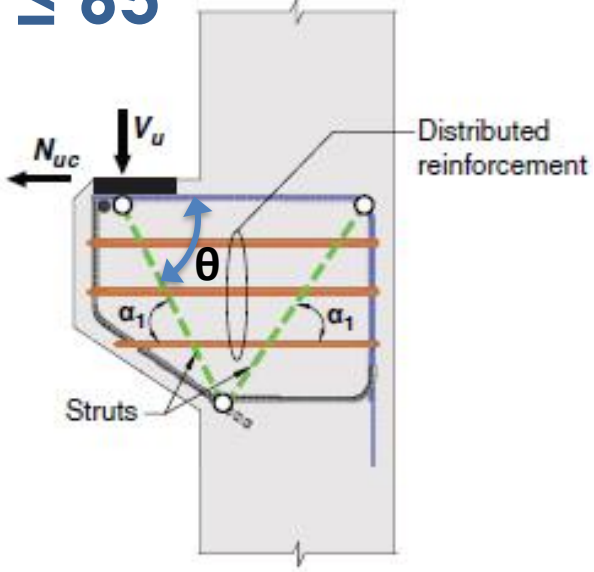
2- $\lambda_s = \sqrt{\frac{2}{1 + d/10}} \leq 1$

23.4 Strength of struts

$$V_u \leq \phi 5 \tan\theta \lambda \lambda_s \sqrt{f'_c}\, b_w d$$

Assume λ = 1, λ_s = 1, and 25° ≤ θ ≤ 65°

tan 65° = 2.14



➔ $V_u \leq \phi 5(2.14)(1)(1)\sqrt{f'_c}\, b_w d$

$\leq \phi 10.7 \sqrt{f'_c}\, b_w d$

Limit to 10 $\sqrt{f'_c}$ consistent with deep beam provision 9.9.2.1



Code Changes—Strut-and-tie method

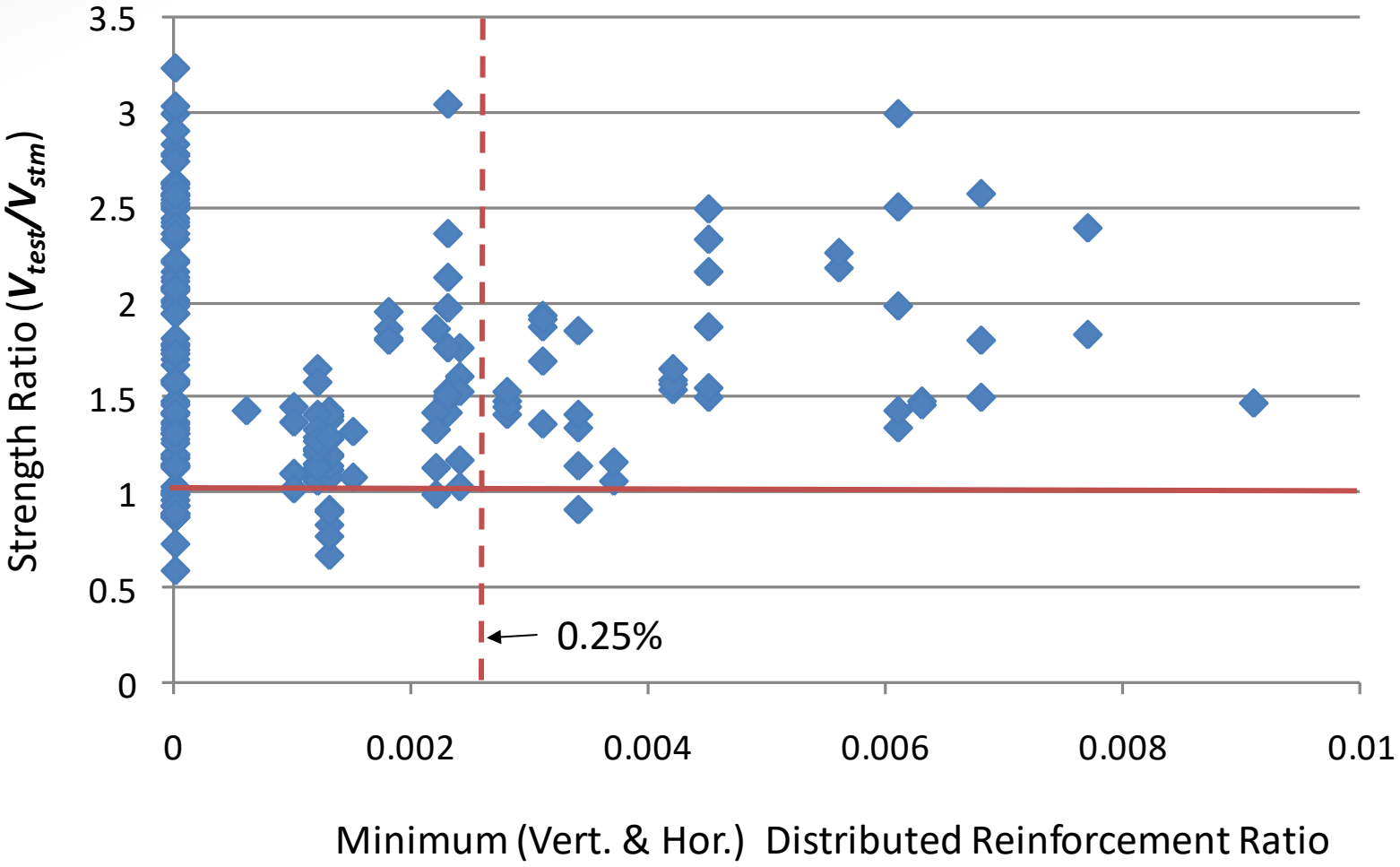
- Minimum angle between strut and tie
- Effect of prestressing
- Development of tie forces
- Strut strength and maximum shear stress
- **Minimum reinforcement in D-region**
- Curved nodes
- STM part of seismic force resisting system

23.5 Minimum distributed reinforcement

ACI 318-19 – minimum distributed reinforcement requirements in deep beams and walls

<table>
<tr><th colspan="2">Member</th><th colspan="2">Distributed reinforcement, ρ_{min}</th><th>Spacing, s</th></tr>
<tr><td colspan="2">Deep beams
(9.9.3.1 & 9.9.4.3)</td><td colspan="2">≥ 0.0025 in each direction</td><td>Min. [d/5 and 12 in.]</td></tr>
<tr><td rowspan="4">Wall</td><td rowspan="3">$V_U \le \phi V_C/2$
(11.6.1)</td><td>Longitudinal</td><td>Transverse</td><td rowspan="4">Min. [3h, 18 in.]
(11.7.2 & 11.7.3)</td></tr>
<tr><td>CIP 0.0012 to 0.0015</td><td>0.002 to 0.0025</td></tr>
<tr><td>Precast 0.001</td><td>0.001</td></tr>
<tr><td>$V_U > \phi V_C/2$
(11.6.2)</td><td>0.0025</td><td>≥ 0.0025</td></tr>
</table>

Minimum Reinforcement of D Regions



23.5 Minimum distributed reinforcement

Table 23.5.1—Minimum distributed reinforcement

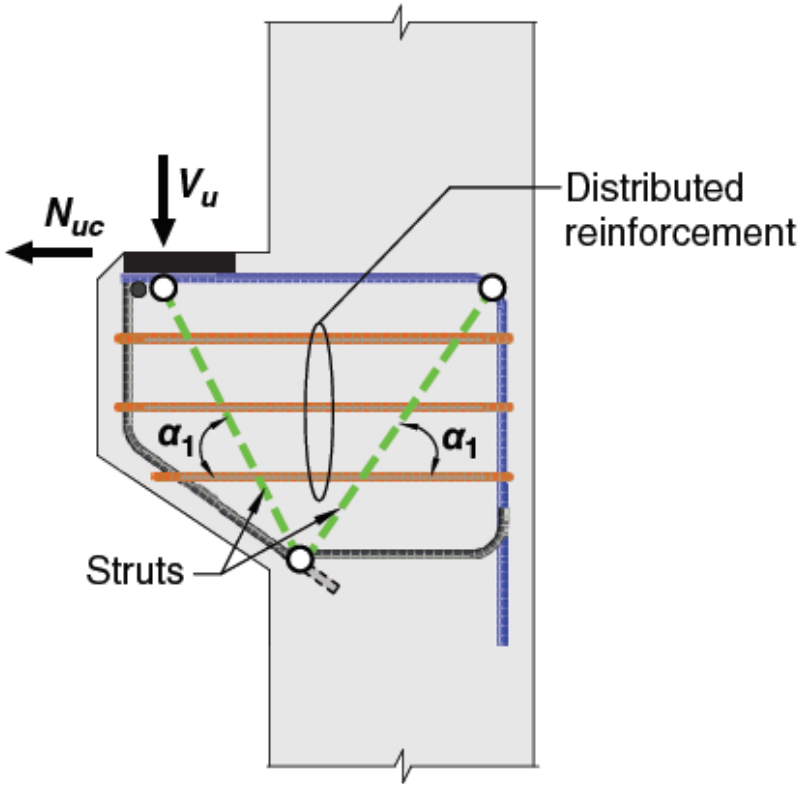
Lateral restraint of strut	Reinforcement configuration	Minimum distributed reinforcement ratio
Not restrained	Orthogonal grid	0.0025 in each direction
	Reinforcement in one direction crossing strut at angle α_i	$0.0025/(\sin^2\alpha_i)$
Restrained	Distributed reinforcement not required	

23.5 Minimum distributed reinforcement

Distributed reinforcement must satisfy:

(a) Spacing not greater than 12 in.

(b) α_1 not less than 40 degrees

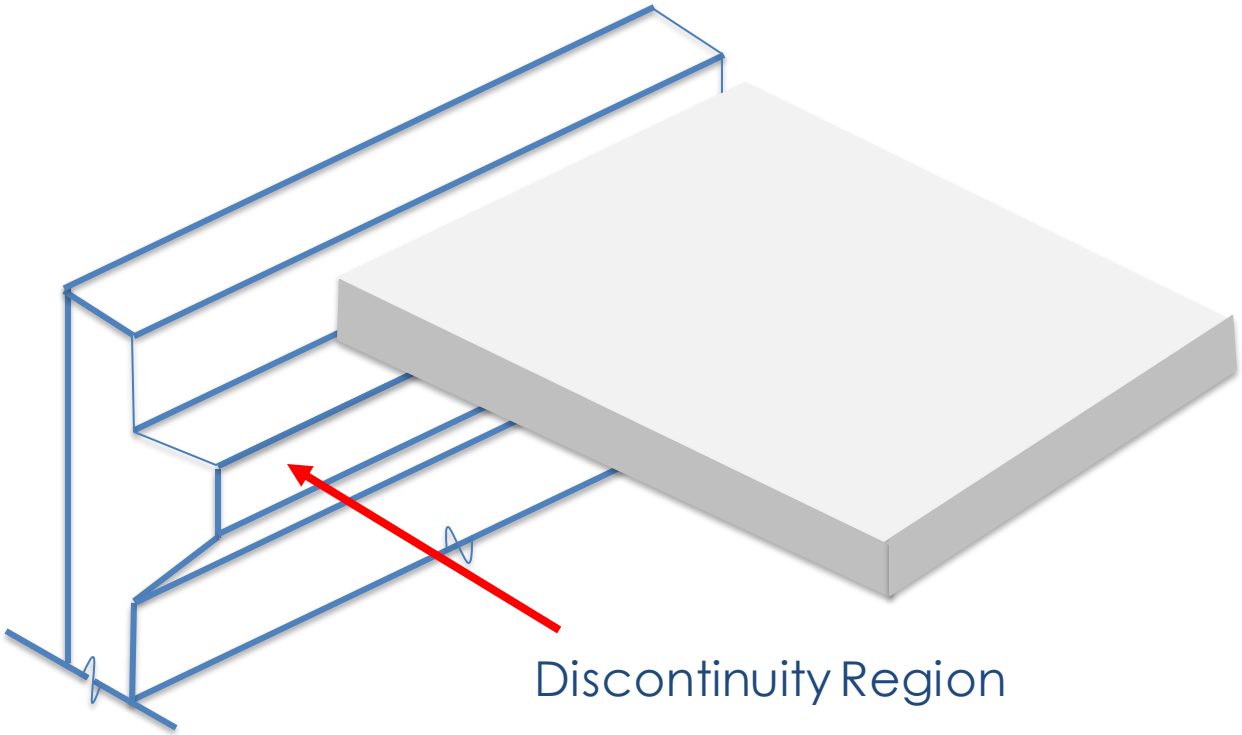


Note: smaller α_1 controls

23.5 Minimum distributed reinforcement

Struts are considered laterally restrained if:

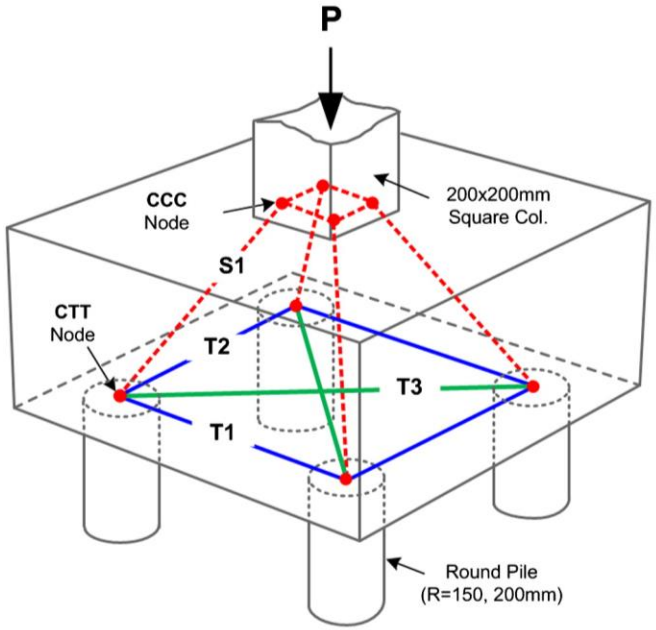
(a)Discontinuity region is continuous ⊥ to plane of STM



23.5 Minimum distributed reinforcement

Struts are considered laterally restrained if:

b) Concrete restraining strut extends beyond each side face of strut a dist. ≥ 1/2 w_s

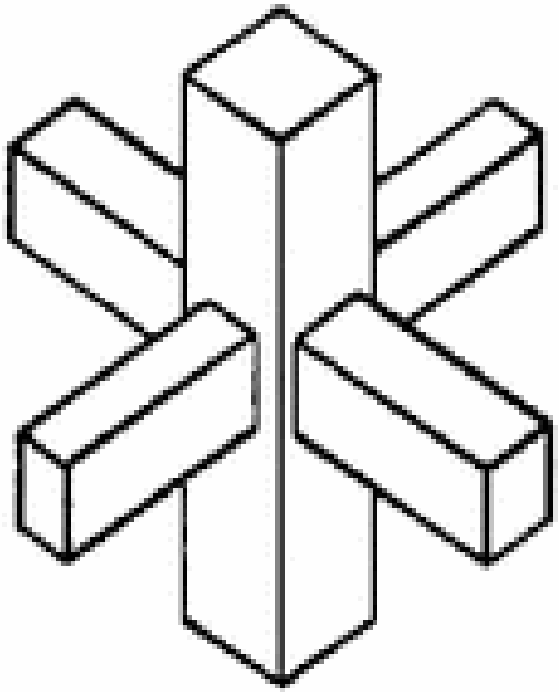


Source: Yun et al. 2016

23.5 Minimum distributed reinforcement

Struts are considered laterally restrained if:

c) Strut in a joint restrained on all 4 faces (15.2.5 & 15.2.6)



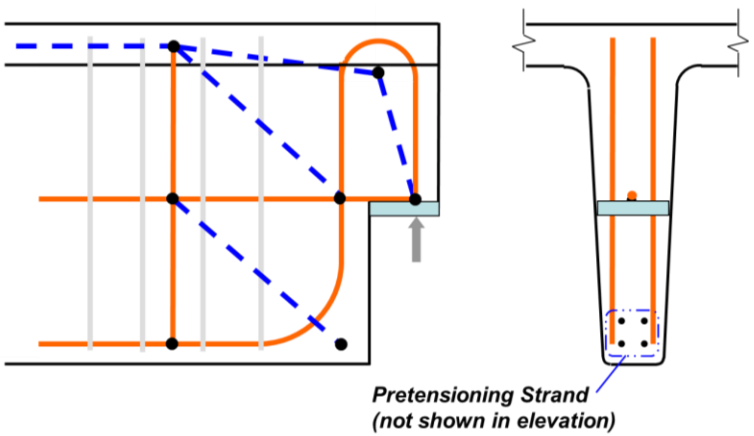
Code Changes—Strut-and-tie method

- Minimum angle between strut and tie
- Effect of prestressing
- Development of tie forces
- Strut strength and maximum shear stress
- Minimum reinforcement in D-region and deletion of bottle-shaped strut
- **Curved nodes**
- STM part of seismic force resisting system

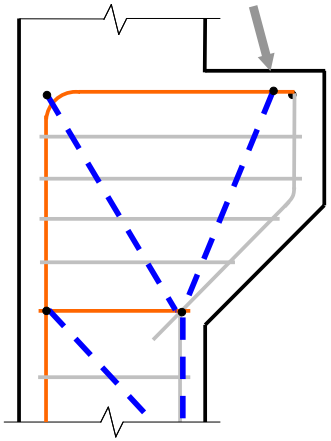
Curved Nodes

Definition

Node, curved-bar – The bend region of a continuous reinforcing bar (or bars) that defines a node in a strut-and-tie model



Dapped-end T-beam

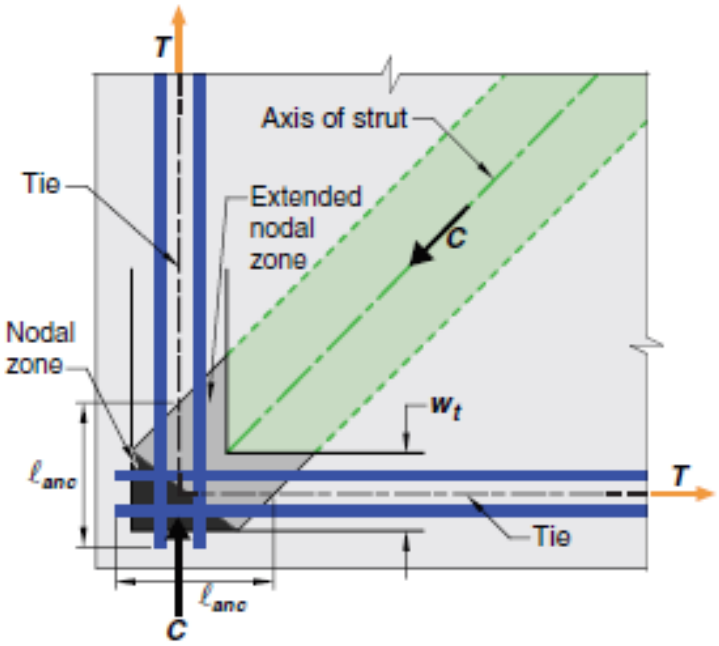


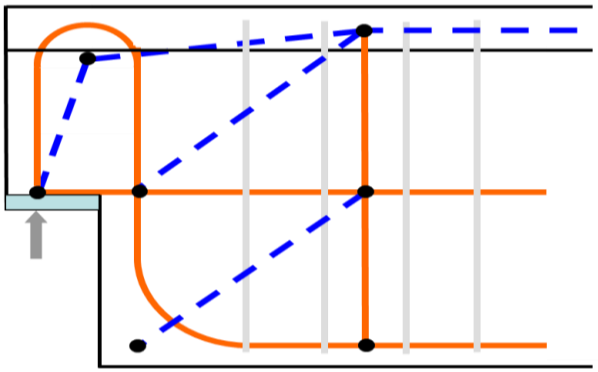
Column Corbel

23.10 Curved-bar Nodes

Why curved nodes?

Nodal zones are generally too small to allow development

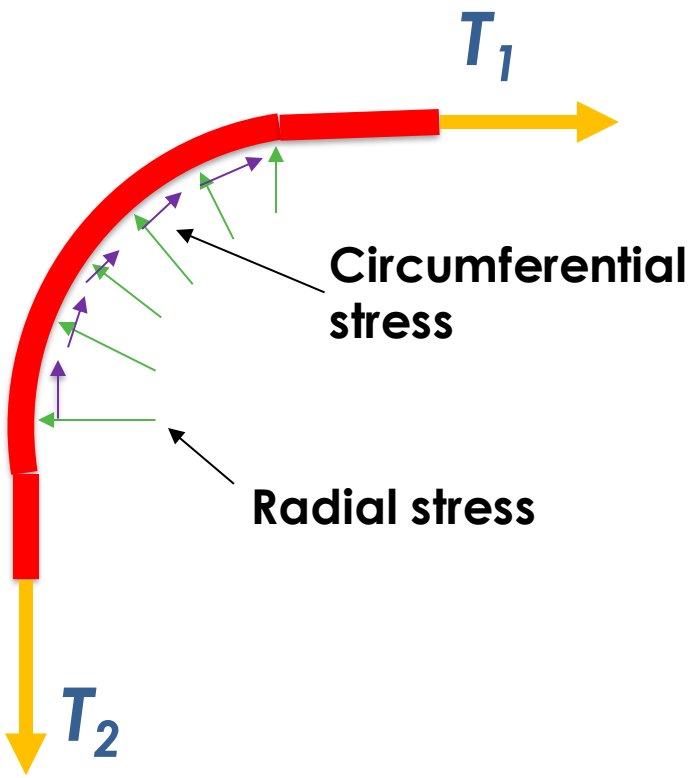




23.10 Curved-bar Nodes

Two issues that need to be addressed:

1. Slipping of bar
2. Concrete crushing



23.10 Curved-bar Nodes

What is the bend radius?

How long is the arc length of the bar bend along centerline of bar?

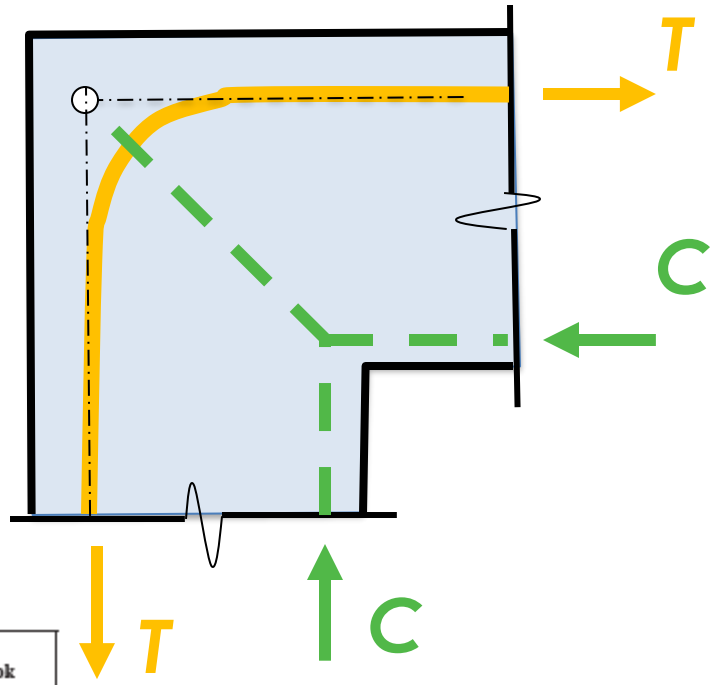
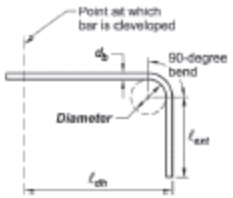
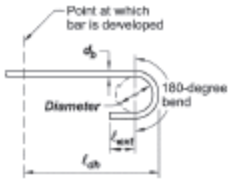


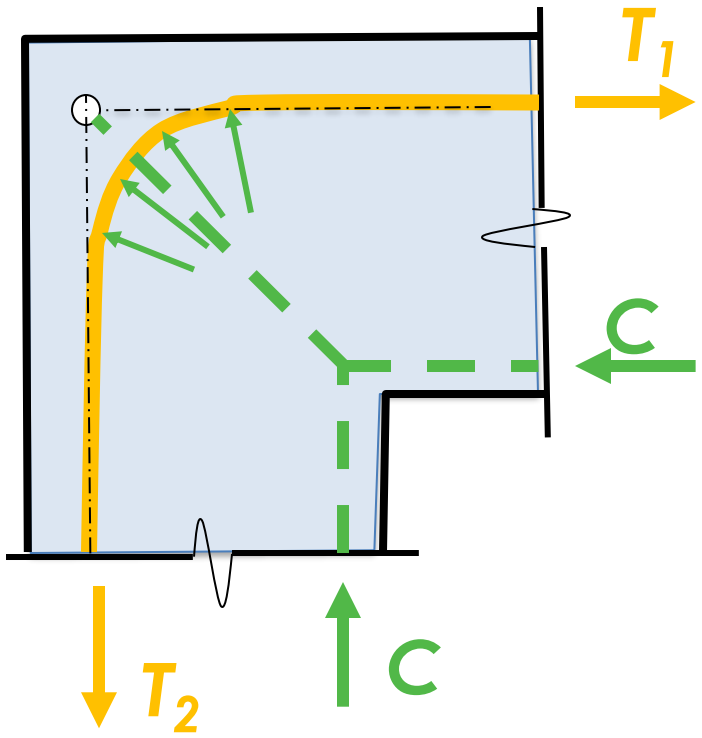
Table 25.3.1—Standard hook geometry for development of deformed bars in tension

Type of standard hook	Bar size	Minimum inside bend diameter, in.	Straight extension[1] ℓ_{ext}, in.	Type of standard hook
90-degree hook	No. 3 through No. 8	$6d_b$	$12d_b$	
	No. 9 through No. 11	$8d_b$		
	No. 14 and No. 18	$10d_b$		
180-degree hook	No. 3 through No. 8	$6d_b$	Greater of $4d_b$ and 2.5 in.	
	No. 9 through No. 11	$8d_b$		
	No. 14 and No. 18	$10d_b$		

23.10 Curved-bar Nodes

1st Condition

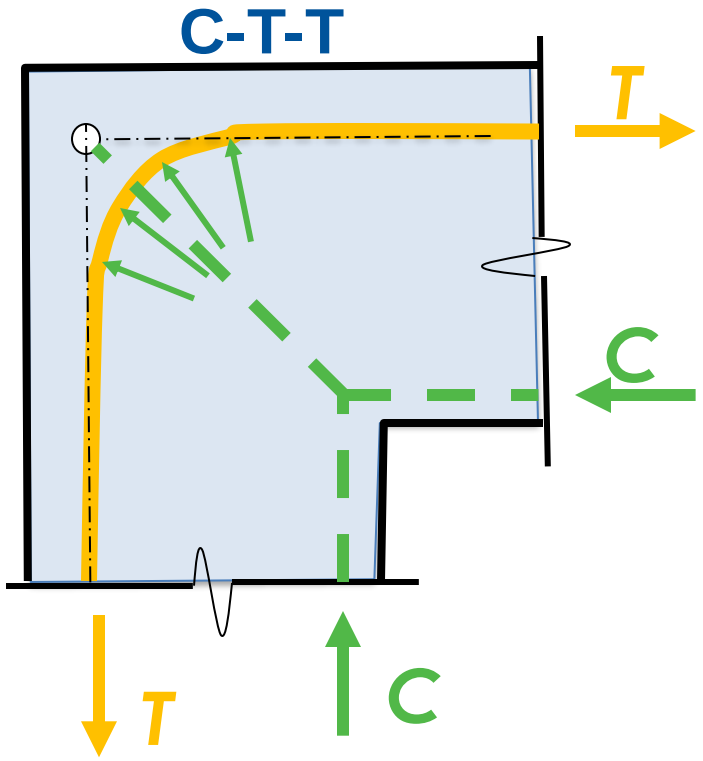
- $\theta < 180$ degree bend
- $T_1 = T_2 = A_s f_y$
- Radial compression stresses are uniform
- Bond stresses = 0



23.10 Curved-bar Nodes

$\theta < 180$ degree bend

$$r_b \geq \frac{2A_{ts}f_y}{b_s f_c'}$$

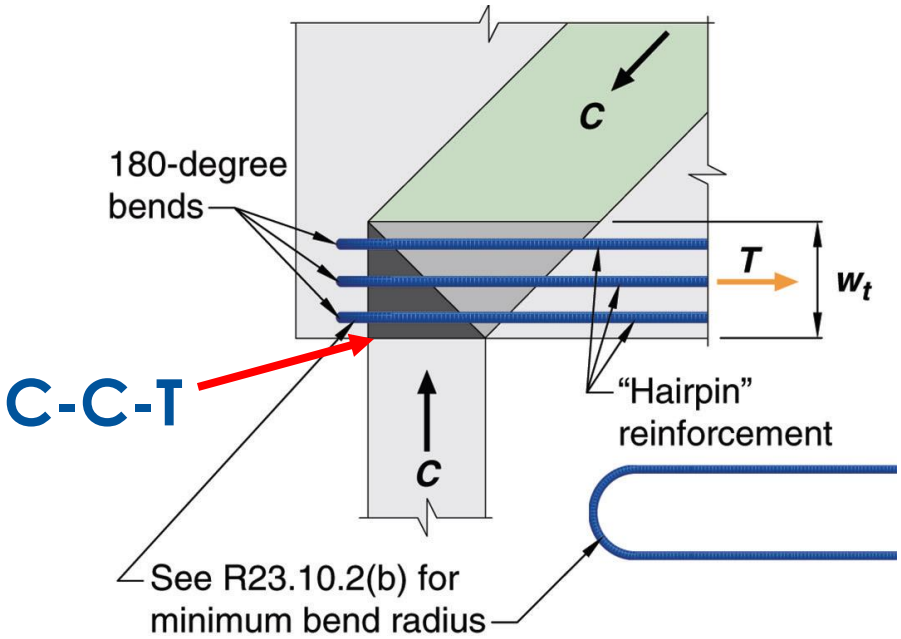


but not less than half bend diameter of Table 25.3

23.10 Curved-bar Nodes

θ = 180 degree bend

$$r_b \geq \frac{1.5 A_{ts} f_y}{w_t f_c'}$$



But not less than half bend diameter of Table 25.3

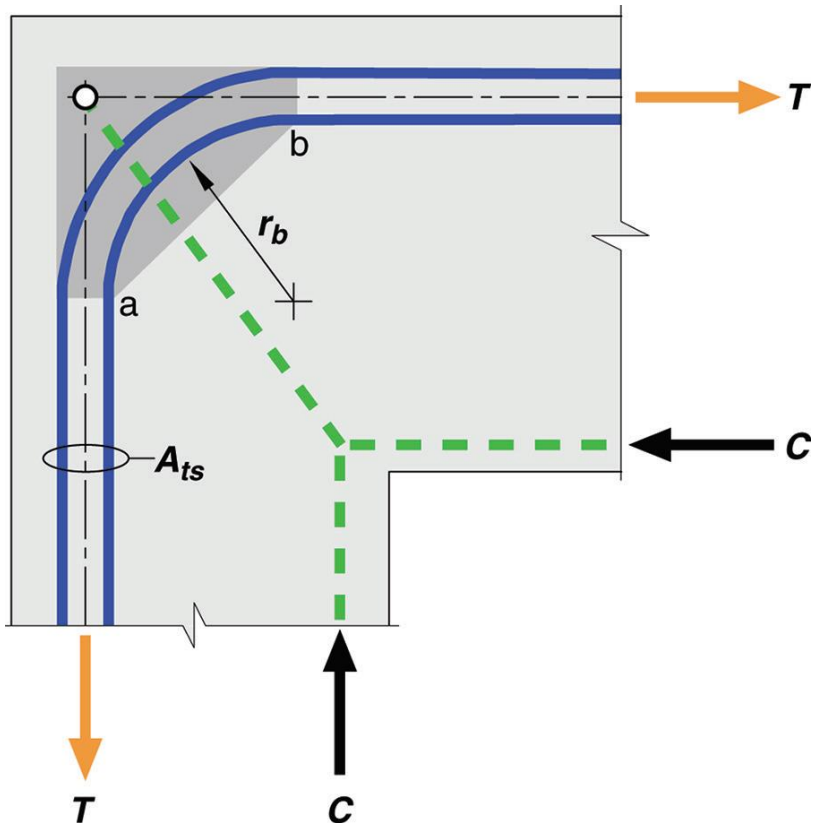
23.10 Curved-bar Nodes

Curved-bar nodes with more than one layer of reinforcement

$$r_b \geq \frac{2A_{ts} f_y}{b_s f_c'}$$

A_{ts} - total area of tie

r_b - radius of innermost layer

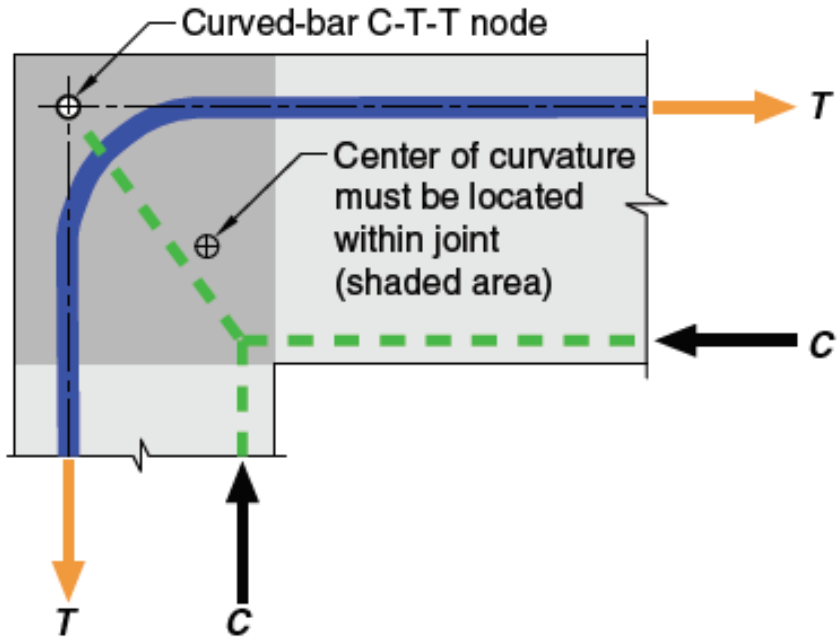


23.10 Curved-bar Nodes

23.10.2 Cover ≥ $2d_b$

23.10.3 cover < $2d_b$

➔ r_b x ($2d_b$ /c_c)



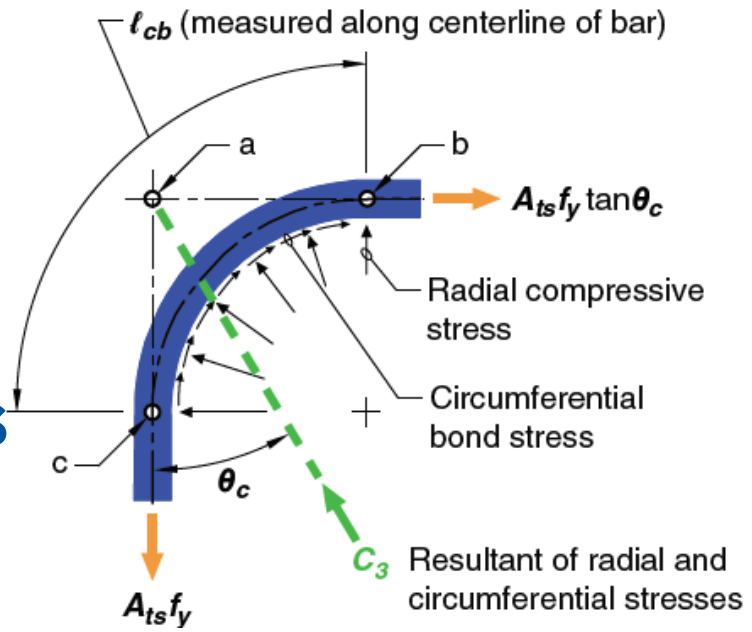
23.10.5 At frame corners, joint and bars are proportioned such that center of bar curvature is located within the joint

23.10 Curved-bar Nodes

2nd Condition

Tie forces are not equal:

- **Compressive stress on the inside radius of bar varies**
- **Circumferential bond stress develops along bar**



θ_c is the smaller of the two angles

$$C_3 = \frac{A_{ts} f_y}{\cos\theta_c}$$

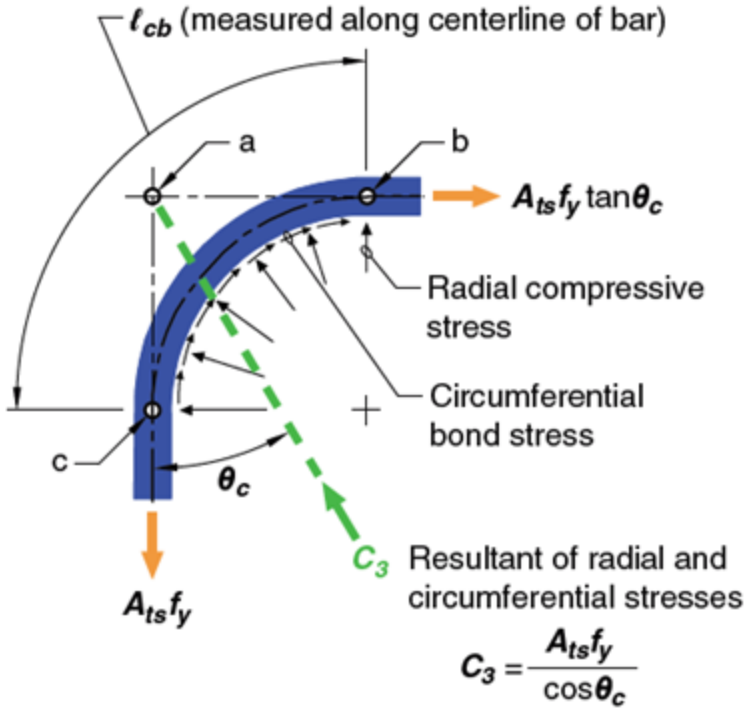
23.10 Curved-bar Nodes

23.10.6 The curve must be sufficient to develop difference in force

$$\ell_{cb} > \ell_d(1 - \tan\theta_c)$$

In terms of r_b

$$r_b > \frac{2\ell_d(1-\tan\theta_c)}{\pi} - \frac{d_b}{2}$$

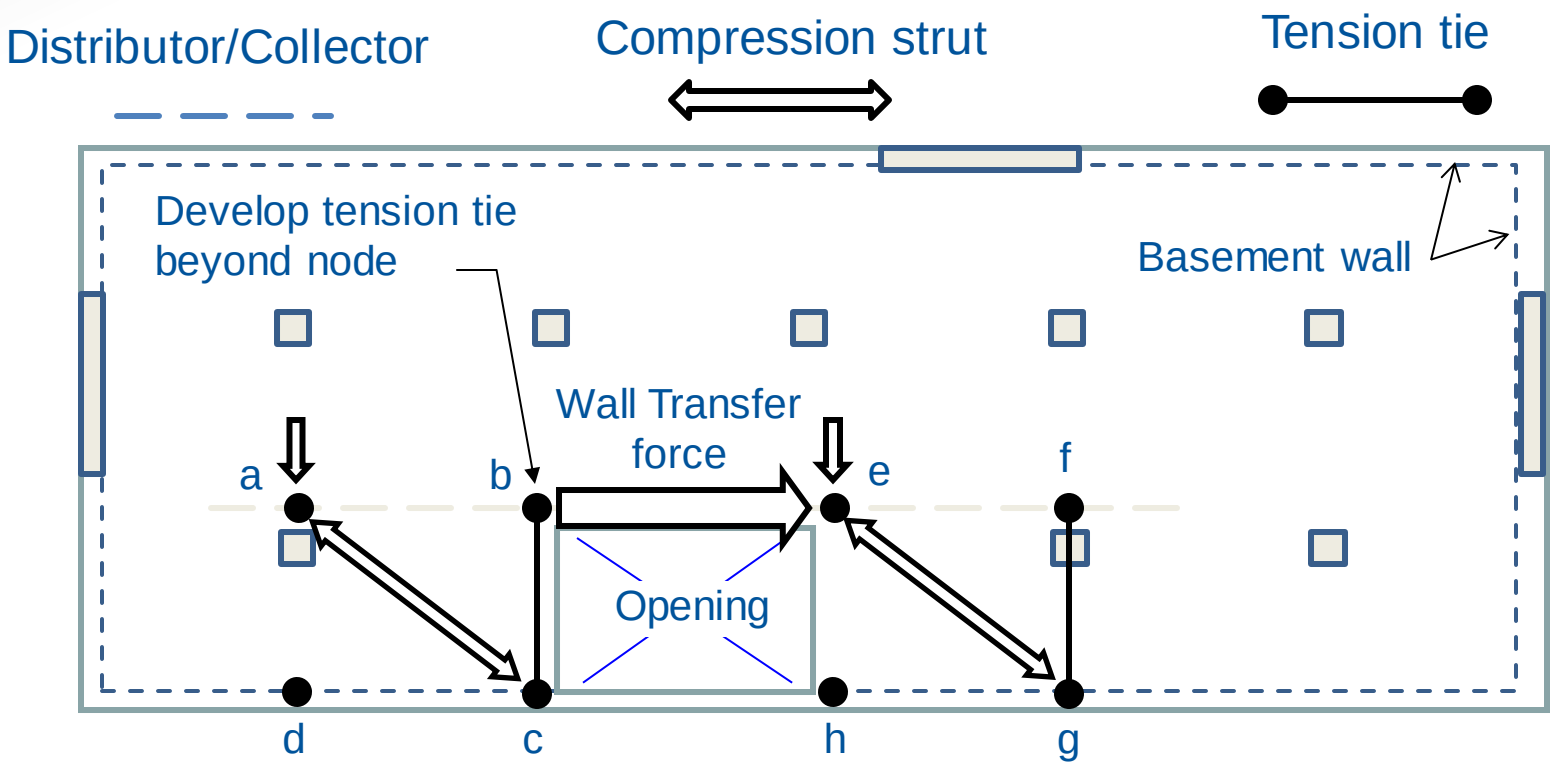




Code Changes—Strut-and-tie method

- Minimum angle between strut and tie
- Effect of prestressing
- Development of tie forces
- Strut strength and maximum shear stress
- Minimum reinforcement in D-region and deletion of bottle-shaped strut
- Curved nodes
- **STM part of seismic force resisting system**

23.11 Earthquake-resistant design using STM





Earthquake-resistant design using STM

Seismic-force-resisting system assigned to SDC D-F and designed with STM must satisfy:

1. Chapter 18
2. Strut forces are increased by overstrength factor $\Omega_o = 2.5$ or $\Omega_o < 2.5$ if based on rational analysis

23.11 Earthquake-resistant design using STM

If condition 2 is not satisfied then the following must be addressed, Provisions 23.11.2 - 23.11.5

1. Provisions 23.11.2 and 23.11.5

   Reduce strut and node effective compressive strength, f_{ce}, of concrete by 0.8

$$f_{ce} = (0.8)(0.85\ \beta_c \beta_{s/n}\ f_c')$$

23.11 Earthquake-resistant design using STM

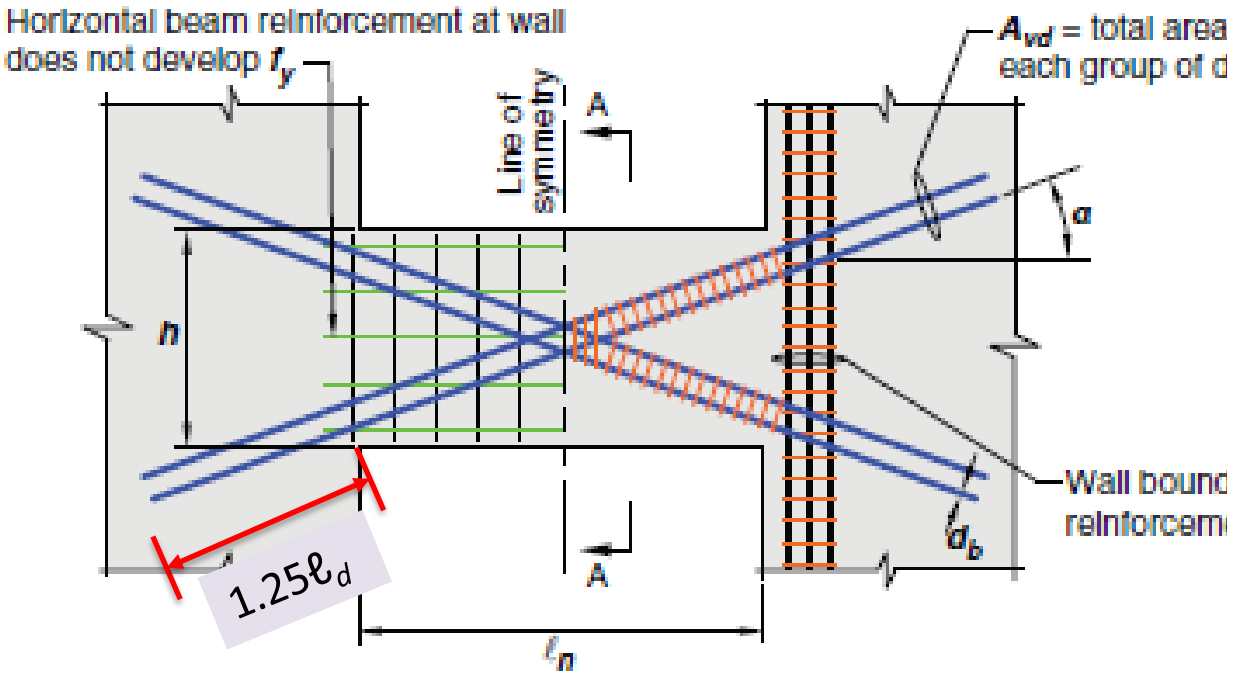
2. Two options for strut detailing, Provisions 23.11.3 and 23.11.4:



- Strut w/min. 4 bars
- Transverse ties perpendicular to strut
- Detailing of ties per Ch. 18 column requirements and Ch. 23 Tables 23.11.3.2 and 23.11.3.3

23.11 Earthquake-resistant design using STM

23.11.4 Tie development length is 1.25 ℓ_d (25.4)



ACI 318-19

American Concrete Institute
Always advancing

Changes to the Concrete Design Standard



Shotcrete

Shotcrete

- Shotcrete equals regular concrete
- Placement method
- Additional information in ACI 506R and ACI 506.2

Table R4.2.1.1—Sections in Code with shotcrete provisions

Topic covered	Section
Freezing and thawing	19.3.3.3 through 19.3.3.6
Reinforcement	25.2.7 through 25.2.10, 25.5.1.6, and 25.5.1.7
Where shotcrete is required or permitted	26.3.1, 26.3.2
Materials	26.4.1.2, 26.4.1.4, and 26.4.1.6
Proportioning mixtures	26.4.3
Documentation of mixtures	26.4.4.1
Placement and consolidation	26.5.2.1
Curing	26.5.3
Joints	26.5.6
Evaluation and acceptance	26.12



Shotcrete

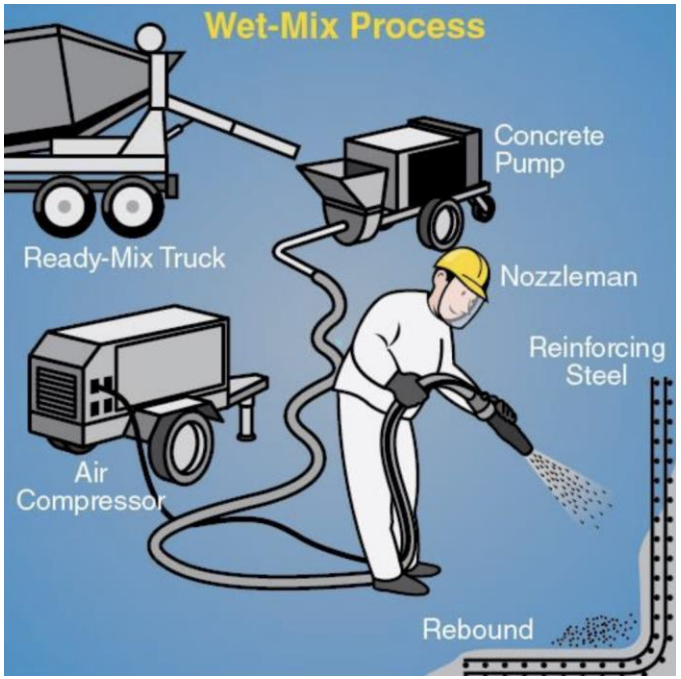
Why Shotcrete?

- **Several applications** – new or repair
- **Economical**
- **Effective**
- **Excellent bond**

Shotcrete

Two processes

- Wet mix
- Dry Mix





Shotcrete

- **Requirements for freezing-and-thawing exposure**
- **19.3.3.3: Air entrainment**
  - **Wet-mix** shotcrete subject to Exposure Classes **F1, F2, or F3**
  - **Dry-mix** shotcrete subject to Exposure Class **F3**
  - Air content shall conform to Table 19.3.3.3.
  - Exception in 19.3.3.6 (similar to concrete)

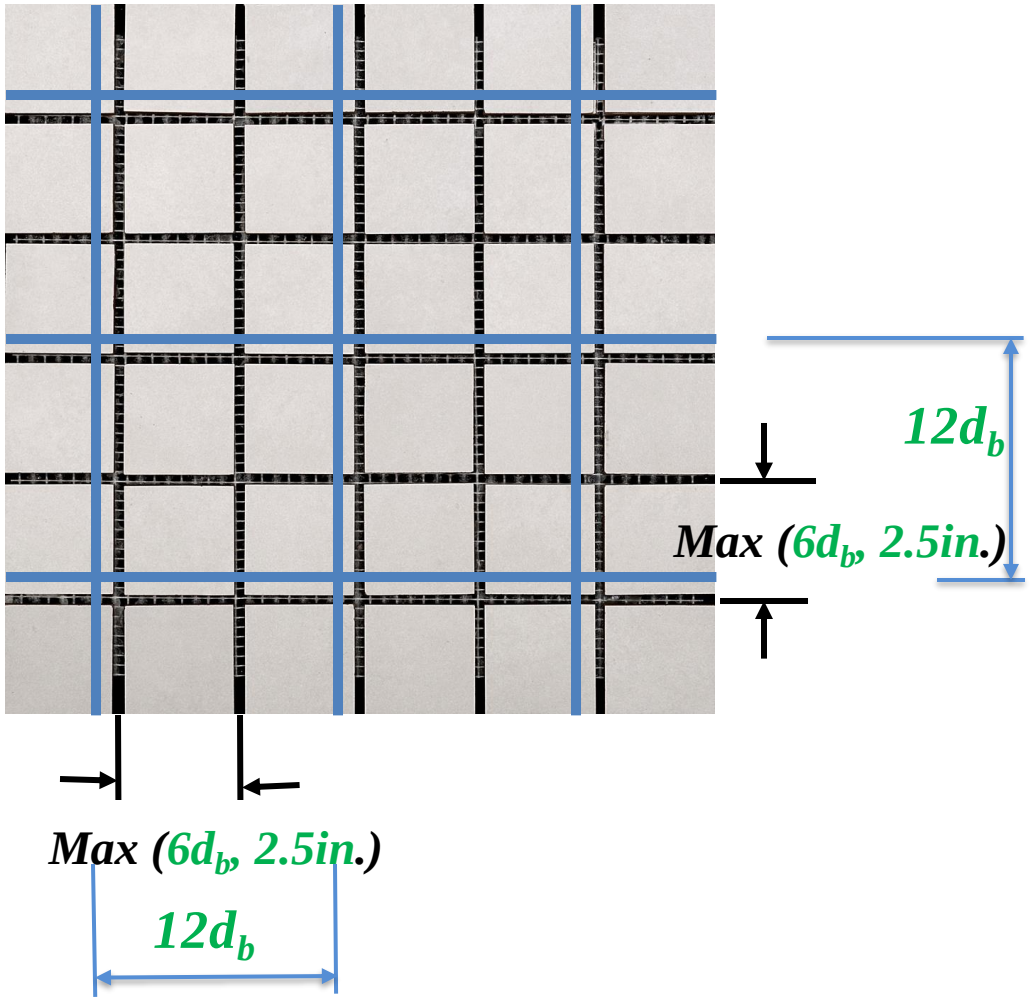
Table 19.3.3.3—Total air content for shotcrete exposed to cycles of freezing and thawing

Mixture type	Sampling location	Target air content, percent		
		F1	F2	F3
Wet-mix shotcrete	Before placement	5.0	6.0	6.0
Dry-mix shotcrete	In-place	N/A[1]	N/A[1]	4.5

[1]Entrained air is not required in dry-mix shotcrete for these exposure classes.

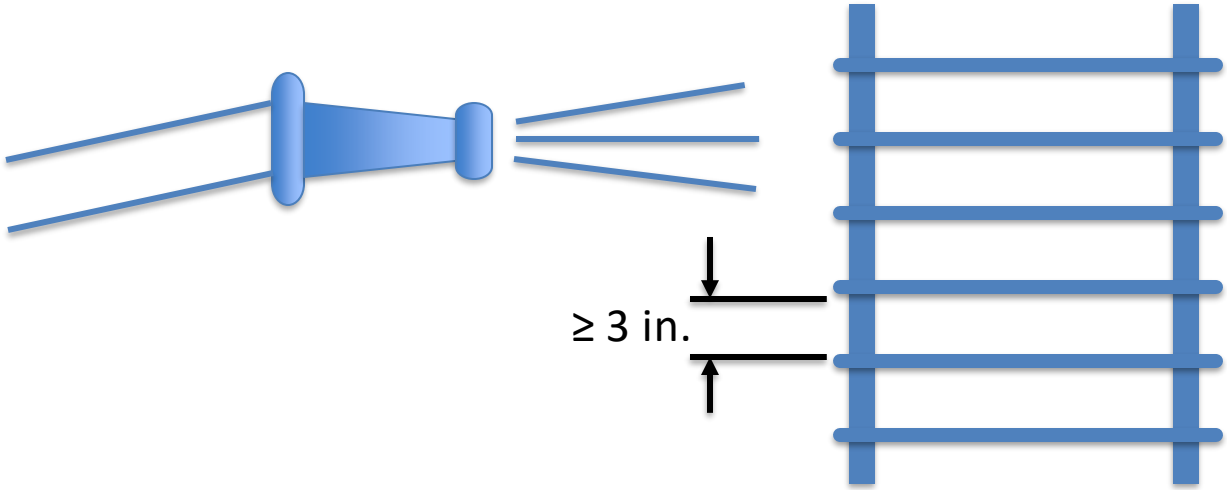
Shotcrete - Minimum Spacing of Reinforcement

- **25.2.7: Parallel nonprestressed reinforcement**
  - (a) at least the greater of **$6d_b$** and **2-1/2 in.**
  - (b) If **two curtains** of reinforcement are provided,
    - At least **$12d_b$** in the curtain nearer the nozzle
    - remaining curtain confirm to (a)



Shotcrete - Minimum Spacing of Reinforcement

- **25.2.10**
  - For **ties, hoops**, and **spiral reinforcement in columns** to be placed with shotcrete, minimum clear spacing shall be **3 in**.



Shotcrete –Splices

- **25.5.1.6 Non-contact lap splices**
  - Clear spacing - No. 6 and smaller bars, at least greater of **$6d_b$** and **2-1/2 in.**
  - Clear spacing - No. 7 and larger bars, use mockup panel
- **25.5.1.7 Contact lap splices**
  - Plane of the spliced bars be perpendicular to the surface of the shotcrete
  - Need approval of the LDP based on a mockup panel



Reinforcement laps

Shotcrete

Mockup panels

- To demonstrate proper encasement of the reinforcement
- Represent most complex reinforcement configurations

Shotcrete

- **Mockup panels**



Mockup panel



Crew shooting mockup panel

Shotcrete

Construction Documents and Inspection

- **26.3.1-26.3.2: Where shotcrete is required**
  - Identify the members to be constructed using shotcrete
- **26.4.1.2 – 26.4.1.7: Materials**
  - Aggregate gradation - ASTM C1436.
  - Admixtures – ASTM C1141.
  - Packaged, preblended, dry, combined materials for shotcrete – ASTM 1480

Shotcrete

- **26.4.2 - Concrete mixture requirements**
  - Maximum coarse aggregate size **≤ 1/2 in.**



Shotcrete

- **26.5.2.1: Placement and consolidation**
  - **Remove rebound and overspray** prior to placement of a new layer
  - Cuttings and rebound shall not be incorporated into the Work
  - **Roughen existing surface to ¼ in**. amplitude before placing subsequent shotcrete
  - Before placing additional material onto hardened shotcrete,
    - **Remove laitance**
    - **clean joints**
    - **dampen surface**

Shotcrete

- **26.5.2.1: Placement and consolidation**
  - **Remove and replace** in-place fresh shotcrete that exhibits sags, sloughs, segregation, honeycombing, and sand pockets
  - Shotcrete nozzle operator
    - must be certified
    - able to shoot an approved mockup panel



Shotcrete

26.5.3: Curing Satisfying (1) – (3)

(1) Initial curing : for first 24 hours

(i) Ponding, fogging, or continuous sprinkling

(ii) Absorptive mat, fabric, or other protective covering kept continuously moist

(iii) Application of a membrane-forming curing compound

Shotcrete

26.5.3: Curing Satisfying (1) – (3)

- **(2) Final curing: After 24 hours**

  (i) Same method used in the initial curing process

  (ii) Sheet materials

  (iii) Other moisture-retaining covers kept continuously moist

- **(3) Maintain final curing**

  for a minimum duration of:

  – 7 days

  – 3 days if either a high-early-strength cement or an accelerating admixture is used

Shotcrete

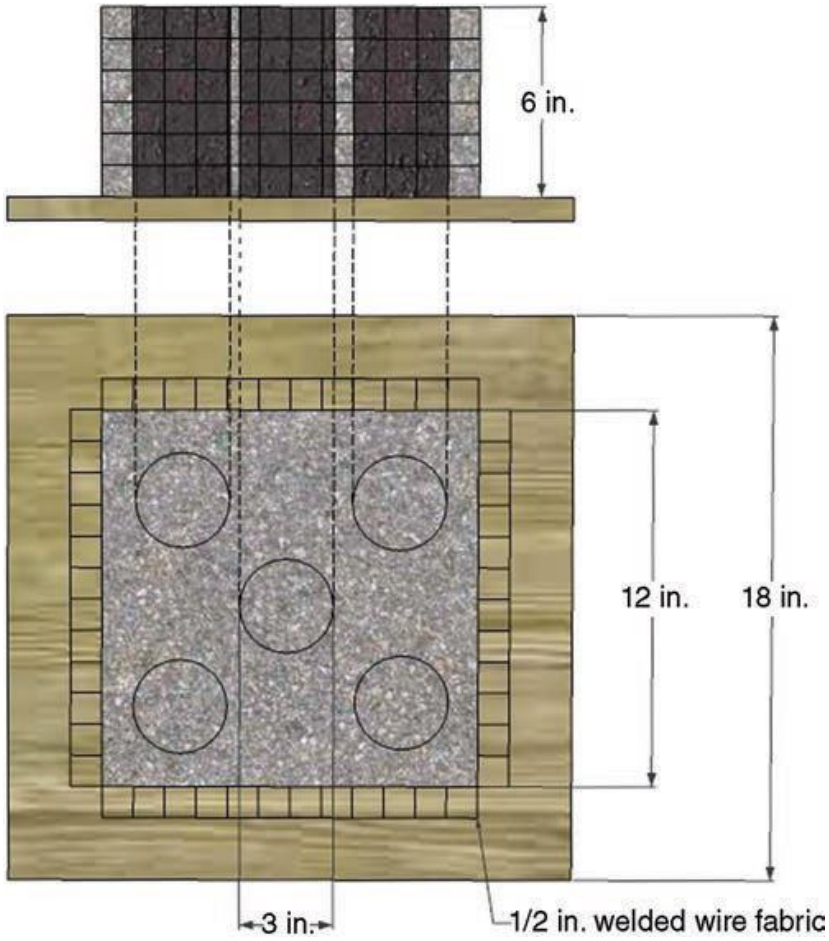
26.5.6: Construction~~, contraction, and isolation~~ joints

- **cut at a 45° unless a square joint is designated**
- **Submit locations to LDP for approval**
  - For joints not shown on the construction documents

Shotcrete

26.12—Evaluation and acceptance

- **Strength test**
  - Average strength of minimum **three 3 in.** diameter cores from a test panel
  - Tested at **28 days** or at test age designated for fc′



Material test panel sketch showing where to cut five cores

Shotcrete

26.12.2 Frequency of testing

- **Prepare a test panel**
  - For each mixture
  - For each nozzle operator
  - at least once per day or for every 50 yd^3
    - whichever results in the greater number of panels

Shotcrete

26.12.4 Acceptance criteria for shotcrete

- **26.12.4.1(a): Test specimens to satisfy (1) and (2):**

  (1) Test panels shall be prepared
  - in the same orientation
  - by same nozzle operator

  (2) Cores as per ASTM C1604

Shotcrete

26.12.4 Acceptance criteria

- **26.12.4.1(b): Strength to satisfy (1) and (2):**

  (1) average strengths from three consecutive test panels **≥ f_c'**

  (2) *average compressive strength of three cores from a single test panel* **≥ 0.85f_c'** *and* **no** single core strength **< 0.75f_c'**

Take steps to **increase strength** if not satisfied

Investigate if not satisfied

ACI 318-19

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Changes to the Concrete Design Standard



Design Verification Using Nonlinear Dynamic Analysis

Appendix A – Design Verification Using Nonlinear Dynamic Analysis

Why was Appendix A added to the Code?

- ASCE 7-16 = yes
- LA Tall Building Council = yes
- PEER = yes
- ACI 318-14 = no
- ACI 318-19 = yes
  - Coordinates treatment of concrete

Appendix A – Design Verification Using Nonlinear Dynamic Analysis

What is Design Verification Using Nonlinear Dynamic Analysis?

- Design basis
- Initial design per ACI 318 (Ch. 18)
- Nonlinear software
- Behaviors in model based on
  - Testing
  - Estimated properties

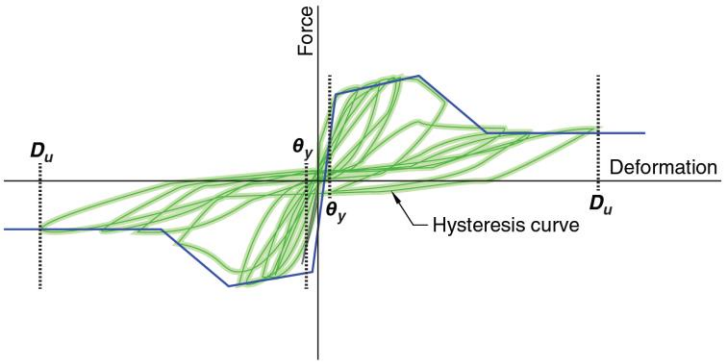


Fig. RA.10.2—D_u in response hysteresis from an analysis model.



Appendix A – Design Verification Using Nonlinear Dynamic Analysis

- Analysis results vs Design basis
- Peer review
- Agreement that structure meets IBC 2018 req.

Appendix A – Design Verification Using Nonlinear Dynamic Analysis

Why would an engineer use Design Verification Using Nonlinear Dynamic Analysis?

- **Tall buildings (over 240')**
  - IBC 2018 ≠ special concrete shear walls
  - Forces dual system
- **Nonlinear Dynamic Analysis**
  - Allows concrete shear walls over 240'
  - Exception per IBC 2018 104.11
- **NOT JUST FOR SEISMIC**

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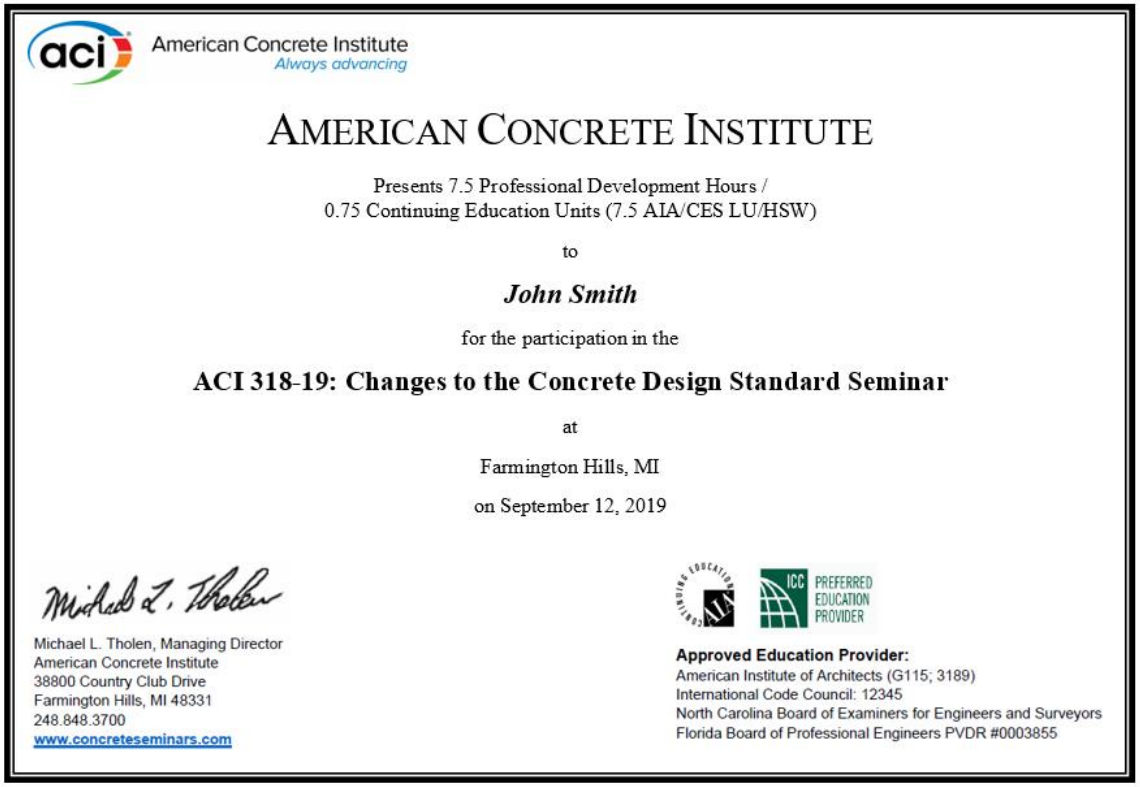
Changes to the Concrete Design Standard



Closing Remarks

Certificates

- emailed to you within 1-2 weeks
- Check email and name on sign-in sheet



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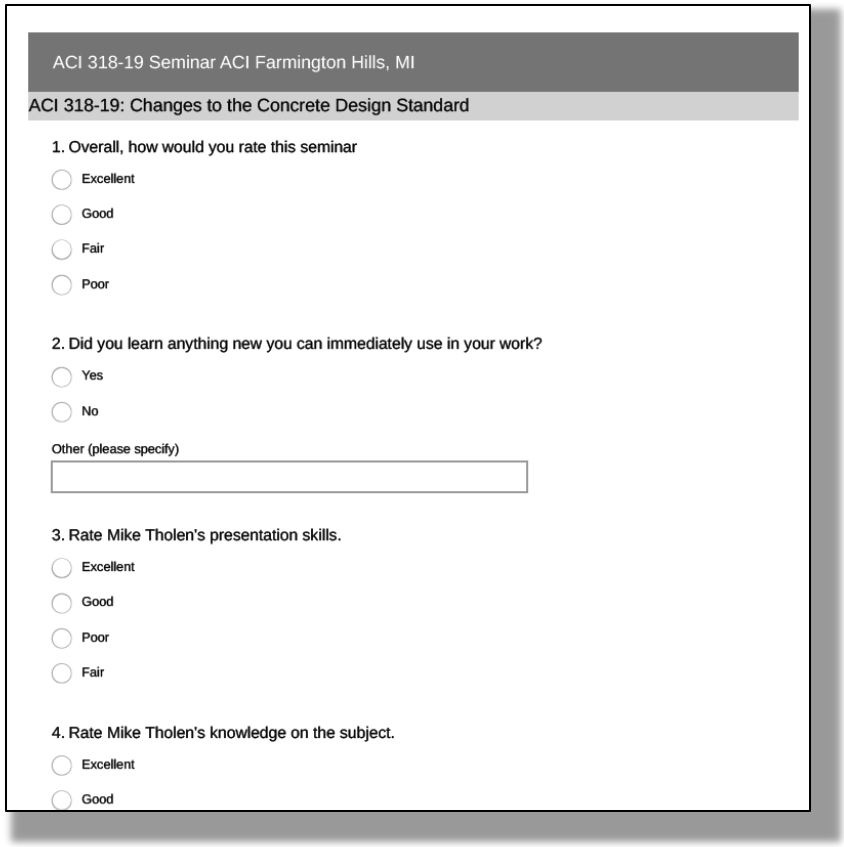
Michael L. Tholen, Managing Director
American Concrete Institute
38800 Country Club Drive
Farmington Hills, MI 48331
248.848.3700
www.concreteseminars.com

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Feedback

- Survey in the email with your certificate
- Brief, 11-question survey


ACI 318-19 Seminar ACI Farmington Hills, MI

ACI 318-19: Changes to the Concrete Design Standard

1. Overall, how would you rate this seminar

- Excellent
- Good
- Fair
- Poor

2. Did you learn anything new you can immediately use in your work?

- Yes
- No

Other (please specify)

3. Rate Mike Tholen's presentation skills.

- Excellent
- Good
- Poor
- Fair

4. Rate Mike Tholen's knowledge on the subject.

- Excellent
- Good

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Thank You